सत्यमेव जयते

Ministry of Road Transport & Highways
Government of India

Capacity Development Project on Highways in Mountainous Regions

GUIDELINE FOR SLOPE PROTECTION AND EMBANKMENT WITH ADVANCED TECHNOLOGY

Part-II Appendix

Published by :
Indian Roads Congress

On behalf of the Government of India,
Ministry of Road Transport & Highways
New Delhi

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Foreword

The road network plays a vital road in the overall development of a country by providing the access to employment, social, health and education. India has a vast area in hilly regions consisting of the Himalayan region from North to North-East, the Central Highlands of Aravali, Vindhya & Satpura ranges, the Sahyadri (Western ghats) and the Eastern ghats. Providing all weather connectivity road network in these hilly regions with extreme climatic conditions, difficult and hazardous terrain, topography and high altitude areas is a tough task. The Ministry of Road Transport & Highways has always taken steps towards the betterment of road transport throughout the country and tackling the problem of roads in hilly regions of the country is one such step.

This Guideline for Slope Protection and Embankment with Advanced Technology, was prepared through the "Capacity Development Project on Highways in Mountainous Regions," bilaterally implemented by MoRTH and JICA. The project started in 2016 and prepared five guidelines that are essential for sustainable development of hill roads in India. Among the five guidelines, Slope Protection and Embankment with Advanced Technology guideline focuses on slope protection and landslide mitigation along hill roads in India and aims to introduce new technologies that would enhance capabilities of Indian engineers in dealing with the challenging issues of landslides along hill roads in our hilly regions. Interestingly, this guideline does not supersede any existing and effective guidelines in India. It aims to supplement Indian guidelines with Japanese technologies which are proven to be effective in Japan and considered effective in India; both countries have similar natural and geological conditions. Especially, technologies of landslide investigation explained in Chapter 1 are essential for improving design, construction, and operation maintenance of hill roads including slope protection and landslide countermeasures.

This guideline also focuses on the formidable phenomena of gravity deformation of bedrock slope in hilly regions. It pointed out that gravity deformation is considered as one of the major reasons of the problematic sinking zones in Indian hill roads, which pose serious burdens on our road maintenance organizations in hilly regions. The cutting edge knowledge and information of gravity deformation are highlighted in this guideline, which are also new to Japan as well. By understanding the phenomena of gravity deformation through this guideline and having actual experiences on sites, engineers in India would find out effective solutions against the sinking zones of Indian hill roads and share it with Japanese engineers in future. This guideline also incorporates the valuable experience of the pilot projects under this technical cooperation project, which provide actual and practical information and thus help the readers properly understand the knowledge and information in the guideline.

The Japanese team as well as the Indian team, especially the Experts from JICA, Counterpart Officials from MoRTH, NHAI & NHIDCL, the Project Director and the Project Team, are to be complimented for their commendable effort in producing these guidelines. Their contribution would go a long way towards the sustainable development of roads in the hilly regions of the country.

The 27th day of January 2022

(I. K. Pandey)
Director General (Road Development) & Special Secretary
Ministry of Road Transport & Highways
Government of India

PREFACE

India is a vast country with diverse geological, climatic regions. Roads are one of the important elements of transport infrastructure for land surface movement of passengers, goods, ammunition & army –intercity, intracity, rural areas and neighboring counties. Indian road network passes through different terrains including hills and mountains. Mountainous regions consist of Himalayan region from North to North-East, the Central Highlands of Aravalli, Vindhya and Satpura ranges, Western and Eastern Ghats.

These mountainous regions, generally, have extreme climatic condition, difficult and hazardous terrain, topography and vast high-altitude areas. Roads in mountainous areas are often composed of multiple civil engineering structures including bridges and tunnels, and proper maintenance is essential to maintain the road function as a network. The Hill regions including the road networks are affected more frequently by floods due to torrential rainfall, landslides, snow avalanche etc. which compel many sections of road network closed during rainy season and winter season. However, accountability for providing safe, reliable and user comfortable road network ultimately rests with the Government departments.

For hill roads IRC has published numerous guidelines, manuals, codes, etc viz IRC: 52, IRC:48, IRC: SP:91; IRC: SP:108: IRC: SP:113, IRC: 5; etc for guidance and adoption by highway professional & road departments.

Ministry of Road Transport & Highways (MORTH), took up the initiative to frame Guidelines for Planning, Slope protection, Mountain bridge, Mountain tunnel & Operation and maintenance of highways in mountainous regions through technical cooperation from Japan International Cooperation Agency (JICA) in 2016. JICA took site visits of hilly roads in India, collected site problem statement, interacted with Indian experts from MORTH, NHAI, NHIDCL, PWD, IRC, etc and also conducted seminars, webinars, meetings, etc before finalization of following 5 documents;

- Guidelines for Planning
- Guidelines for Slope protection and embankment with advanced technology
- Guidelines for Mountain bridge with advanced Technology

- Guidelines for Tunnel on Arterial Roads in Hill Areas
- Guidelines for Operation and maintenance

The Guidelines for Highways in Mountainous Regions, as prepared by JICA mainly highlights appropriate planning, design principles for promoting safety conscious planning and design of roads, embankment, slope & its operation & maintenance aspect and will go a long way in guiding the highway engineers and planners to make our, hill roads, tunnels, bridges, safer, comfortable and of international standards.

It is quite natural, this JICA publications cannot be expected to cover all the subjects and details of highway engineering and as such should not be considered as a substitute for the Standards and Specifications as well as Codes of Practice.

The intent of this publications is to provide guidance to the engineers with the critical aspects of development of Highways in Mountainous Regions. These guidelines are dynamic in nature and the users, based on their experience, give appropriate feedback so as to enable the Guidelines to be revised/improved from time to time.

(S K Nirmal)
Director General (Road Development) and Special Secretary
Ministry of Road Transport and Highways &
Secretary General, IRC

Dated July, 2022

GUIDELINE FOR SLOPE PROTECTION AND EMBANKMENT WITH ADVANCED TECHNOLOGY

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Appendix I

Necessary Survey for Hill Road

(Draft) Surveys Necessary for DPR Preparation of Hill Roads

1 Introduction

In this section, types and quantities of surveys necessary for DPR preparation of hill roads are explained. The surveys for tunnels or bridges for hill roads are, however, summarized in the tunnel or bridge guideline respectively. Thus those guidelines should be referred for this purpose.

Firstly, photo or map reading should be performed. Based on the results of photo or map reading, necessary surveys are discussed for the following cases; the Normal Case without gravity deformation or landslide, and the Special Case for gravity deformation or landslide blocks interpreted in map reading or identified by existing information, which interrupts the planned hill roads to be studied in the DPR.

The necessary surveys for the Special Case can be applied when landslides affect the existing hill roads in service as well.

The term "landslide" in this document, when it is used without any modification or limitation or explanation, includes landslide with landslide clay on slip surface, rock fall/slide, slope failure, and deep-seated landslide, but excludes gravity deformation.

2 Photo or Map Reading

2.1 Purpose

Photo or map reading should be done to detect hazardous blocks of gravity deformation or landslide lying on the planned green field alignment or widening of hill roads.

Photo or map reading provides essential information of topographic and geological conditions in hilly regions, which can be used for the surveys for mitigation or countermeasures against landslides on existing hill roads in service. Thus use of photo or map reading for such surveys is recommended as well.

Chapter 1.5.3 of the Slope Guideline describes photo or map reading practice.

2.2 Verification by Field Survey

Interpretation through photo or map reading is basically preliminary and should be verified through field surveys of the actual site.

2.3 Timings

Gravity deformation or landslide blocks on the alignment of hill roads govern the contents and quantity of the surveys to be done in DPR preparation. Thus photo or map reading should be done prior to TOR/RFP preparation for procurement of a DPR Consultant. If the authorities do not exercise photo or map reading, it should be outsourced separately from procurement of a DPR Consultant.

2.4 Areas

Photo or map reading should be performed for the areas up to the ridge lines of the catchment areas along the alignment of hill roads to be surveyed.

2.5 Scale

The scale of photos or maps to be used for photo or map reading are around S=1:10,000 ~ 25,000.

3 Topographic Mapping and Geological Field Survey (Normal Case)

3.1 Purpose

Topographic mapping and geological field survey are essential for design of hill roads as well as countermeasures against landslides including slope failure and rock fall.

Topographic mapping and geological filed survey compose major parts in DPR and are to be incorporated in a TOR for procurement of DPR Consultant.

3.2 Areas

For hill road construction, areas of topographic mapping should be more than ROW so that necessary areas for cut / fill slope as well as construction of landslide countermeasures including temporary works for transport and storage of materials could be surveyed.

The areas to be mapped and surveyed should cover ROW as well as the areas of 300 meters

widths from the planned shoulder of cut slopes and toe of fill slopes. Cross sections for cut / fill slope should be prepared for the same widths of topographic map.

Since the areas of high and long cut sometimes requires survey and mapping up to the ridge lines of hill slopes along the roads, the authority should reserve a right to add areas of mapping and surveying at its own discretion for such areas.

3.3 Requirements

In addition to the requirements of ordinary TOR for DPR consultants, additional requirements should be added for DPR of hill roads.

For topographic mapping, the following requirements should be added:

- Detailed mapping including profile / cross section should show features of landslide / gravity deformation / failures as well as source of rock fall.
- Topographic features listed in Table 1-4 of Chapter 1.1.3 of the Slope Guideline should be mapped precisely.

For the practice of topographic mapping, the following requirements should be added:

- Contour map should reproduce actual shape of terrain as much as possible.
- Control points, reference points, temporary reference points, traverse points, and surveyed points on maps and cross sections should be shown on the ground with appropriate stakes, pins, and markers on stable locations.
- Cross section should be surveyed placing a total station and mirrors right on the survey line; deviation more than 10 cm is not allowed.
- Boring locations as well as survey lines for geophysical survey should be surveyed and shown on the maps. Stakes should be placed on the site.
- For counter map, coordinates and elevation on 10m grid points (monument or stake is not required), each change point of the terrain, and planimetric features shall be obtained; plan view shall contain particular landforms of landslide, road facilities and structures as well as utilities such as electric poles and cables, water pipes, telephone and communication lines, gas pipe lines and others structures.
- Interval to be measured for cross sections: Approx. 20m intervals and at each change in terrain and structures

For geological field surveys, the following requirements should be added:

- Geological field survey should show geological conditions of the sites, including soil / rock types, groundwater spring, shape and characteristics of head scarp, locations and aperture of open cracks, conditions of crack and fissures, conditions of strata, and weathering conditions etc.
- Geological features listed in Table 1-8 and 1-9 of Chapter 1.1.4 of the Slope Guideline should be checked on site and mapped precisely.

For the practice of geological field survey, the following requirements should be added:

- Geological field surveys should use topographic map prepared in the DPR, the scale of which is usually S=1/2,500 ~ 1/10,000.
- Features of geological conditions should be mapped.
- Results of geological field survey should be organized into geological map and geological profile and cross sections on the centerline and important cross sections as well as site photos to explain geological conditions of the sites.

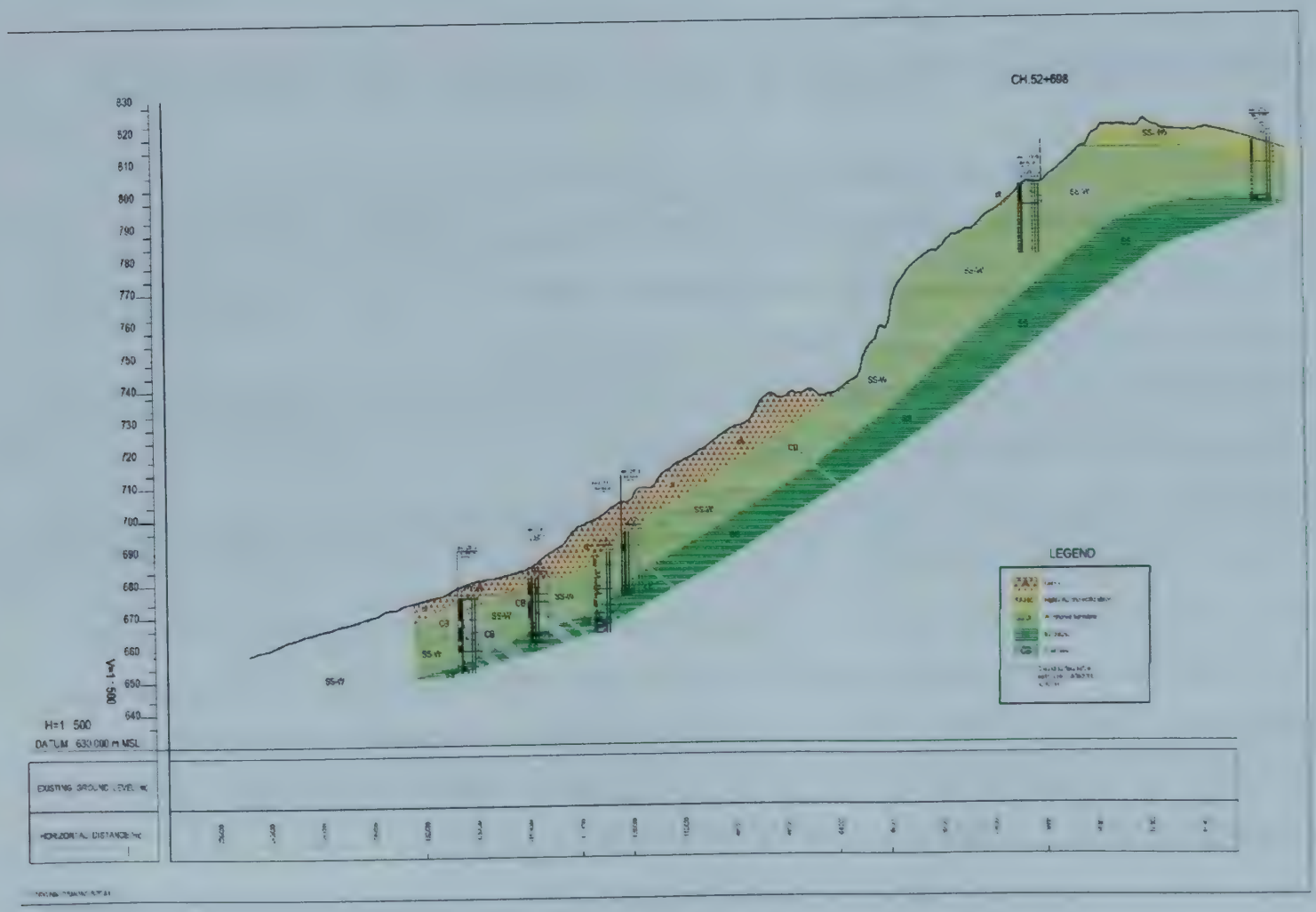


Figure 3-1 Geological Profile (NH55, WB)

Japan International Cooperatino Agency, "Preparatory Survey on the Project for Countermeasure Construction for the Slope Failures on National Highway 55," 2017.

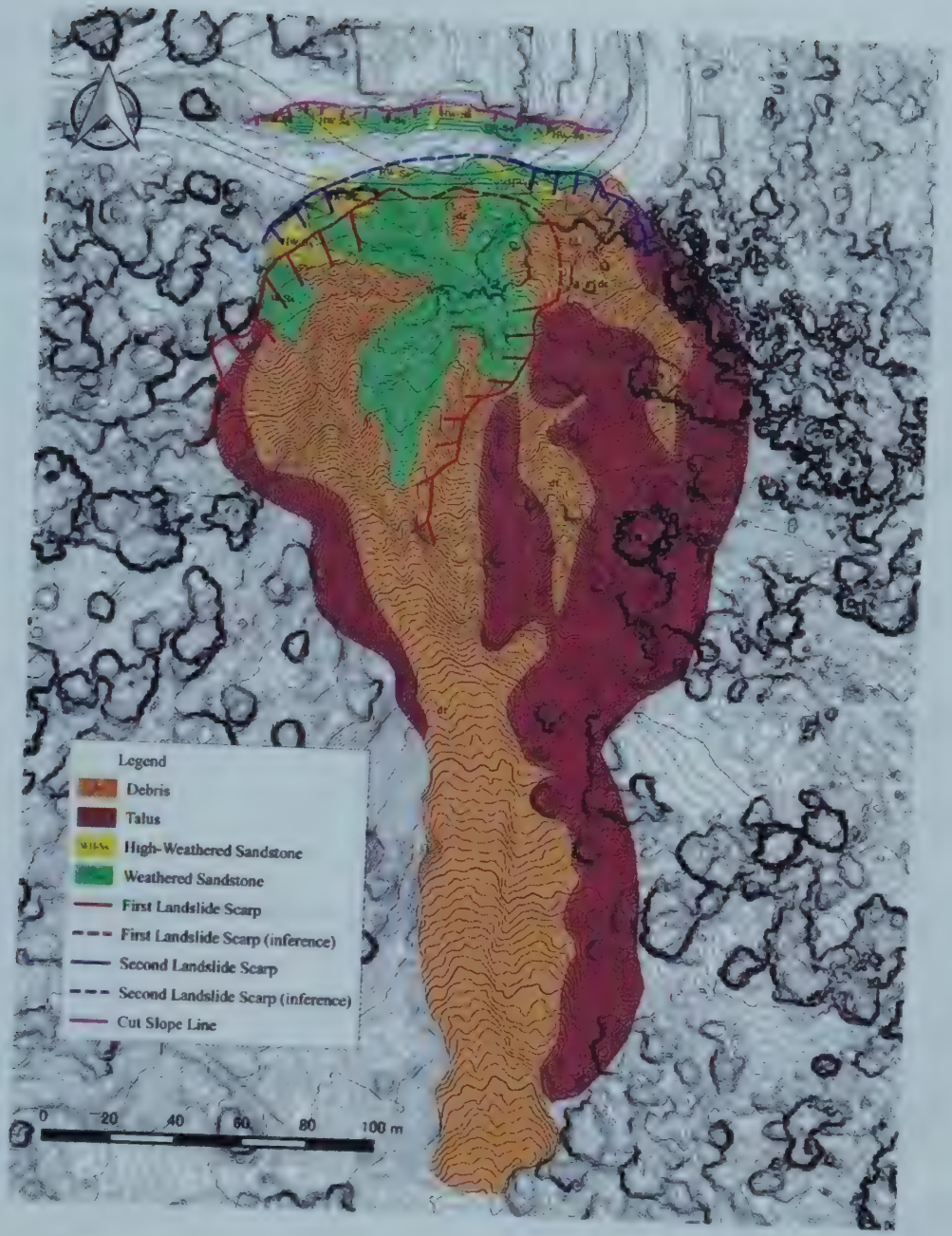


Figure 3-2 Geological Map by Field Survey (NH55, WB)

Japan International Cooperatino Agency, "Preparatory Survey on the Project for Countermeasure Construction for the Slope Failures on National Highway 55," 2017.

3.4 Survey for Debris Flow

In case hill road crosses streams or rivers which may pose threats of debris flow, adequate field survey should be performed. Field survey should be along the streams and rivers to understand erosion condition along stream/river bed and banks as well as slopes along the stream/river. Conditions of deposits on stream/river bed should be surveyed as well, including size of boulders and particle of matrix. The area of survey should be along the stream/river and up to the ridgeline of the catchment areas.

Chapter 1.2.2(2)(iii), 1.2.3(6) and 1.3.2(6) provide details of surveys for debris flow.

4 Drilling Surveys with Core Sampling (Normal Case)

4.1 Purpose

Drilling survey with core sampling provides essential information for design of hill roads as well as stability of cut / fill slopes and design of slope protection, if any. Drilling with core sampling should be conducted by DPR consultant.

4.2 Requirements

In addition to the requirements of ordinary TOR for DPR consultants, additional requirements should be added for DPR of hill roads.

4.2.1 Borehole Locations

(1) Cut Slope

For the section of cutting, drilling surveys with core sampling should be done as follows:

- At least one borehole at each cut slope section.
- For long cut sections, at least one borehole should be planned at every 100 ~ 500m section.
- Drilling location should be on the centerline for through cut. That for sidehill cut should be at the toe of the planned cut slope.
- For cut sections, a drilling should reach the depth of 2 ~3 meters below the planned formation level, as shown in (a) in Figure 4-1. In case hard rock is confirmed for more than 5 meters before reaching the formation level as shown in (b) in the same figure, a drilling may be stopped unless a weak layer is assumed below the formation level, for which a drilling should confirm the depth below the weak layer as shown in (c) in the same figure.
- For high cut sections, where the height of cutting exceeds 20 meters or the cut slope has more than 3 berms placed every 7 meters, drillings on the hill side should be considered as shown in (d) in Figure 4-1, to confirm geological conditions including weathering, cracks and joints, and weak layers (fractured or loosened zones or slip surface) in the cut slopes.
- SPT should be performed at every one meter for overburden and weathered rock as well as weak layers where encountered. SPT should be performed until SPT N-Values of more than 50 are recorded for consecutive 4 tests. When weak layers are

encountered in deeper sections, SPT should be performed.

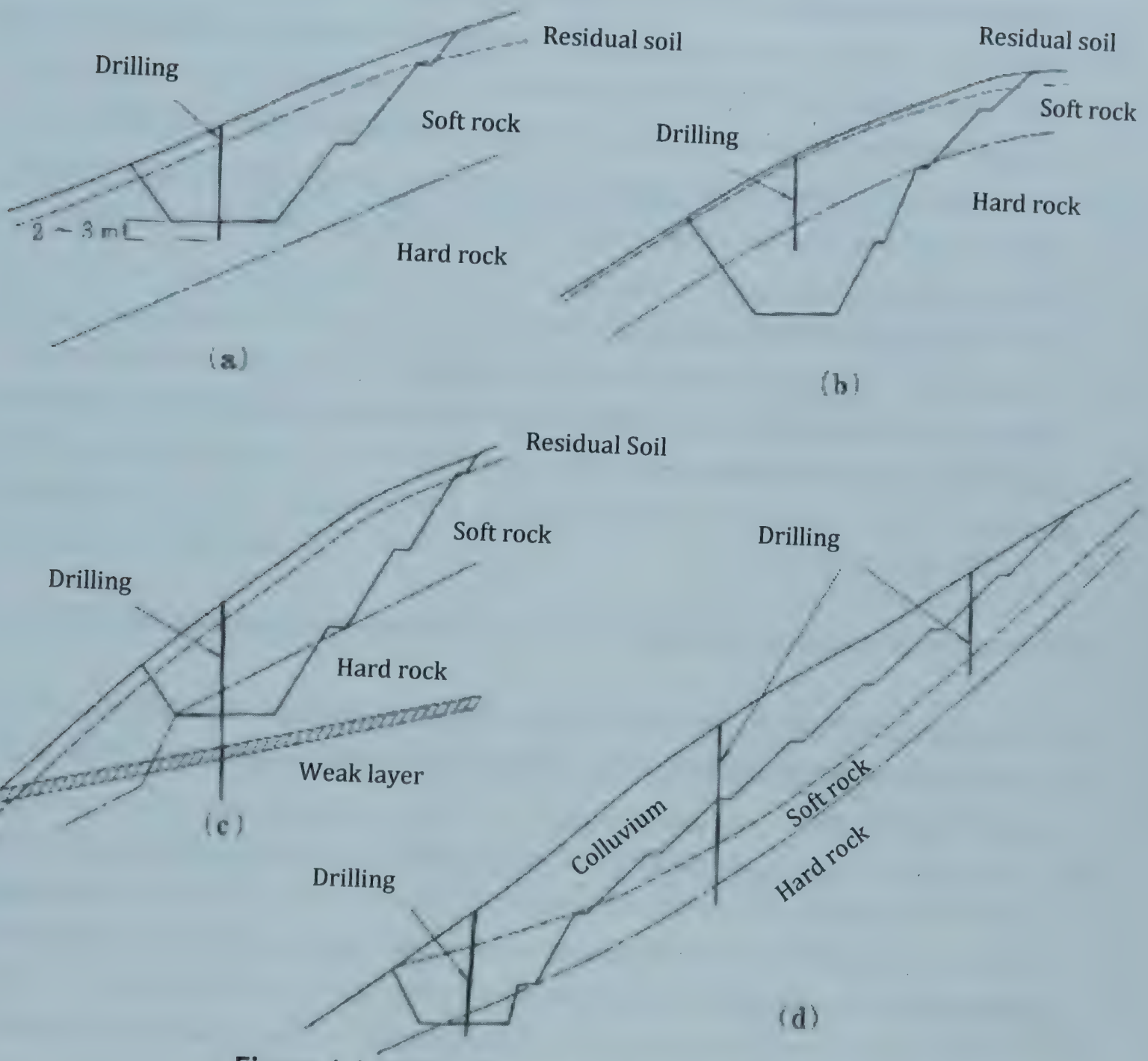


Figure 4-1 Drilling Planning for Cut Slope

Japan Road Association, Highway Earth Work Series-Manual for Slope Protection, Japan Road Association, 2009.

(2) Fill Slope

For the section of filling, drilling surveys with core sampling should be done as follows:

- For short filling sections of a stretch of less than 200 ~500 meters, where soft ground is anticipated or filling is on hill slopes, at least 3 boreholes are required.
- For long filling sections of a stretch of 300 meters or more, where soft ground is anticipated or filling is on hill slopes, at least one borehole should be planned at every

200 ~ 500m section.

- For a filling section crossing a buried valley, at least one bore should be planned.
- Boreholes should be planned on the center line.
- For filling sections with a filling height of 15 meters or higher, additional boreholes should be planned on the planned locations of the toe of fill slope. If such high embankment is planned on hill slopes, additional boreholes should be on valley side.
- For filling sections where soft ground is anticipated, undisturbed sampling for soft ground should be planned using an additional borehole just besides the borehole for SPT and coring. .
- SPT should be performed at every one meter.
- Drilling should reach the depth where SPT N-values of more than 50 are recorded for a consecutive section of 5 meters deep; for such sections, at least 6 SPT N-values of more than 50 should be recorded.

(3) Structure

For the section of structures, drilling surveys with core sampling should be done as follows:

- At least one borehole on the center line should be planned for a structure.
- If the planned road is 4 lanes or wider, additional boreholes at both sides of a structure should be planned.
- If a structure has a long stretch along the road like retaining walls, additional boreholes should be planned at both ends of the structure. If a structure is longer than 50 meters, in addition to the boreholes at both ends, additional boreholes should be planned at every 50 meters along the structure.
- For SPT and undisturbed sampling as well as depths of borehole, same requirements with fill slopes should be applied.

4.2.2 Diameter of Boreholes

Borehole diameter should be enough large to enable in-situ tests including SPT, core sampling, undisturbed sampling, and setting tubes or pipes for monitoring, such as monitoring wells, pipes for borehole inclinometers, and pipe strain gauges. Basically the diameter of borehole should be at least NW/NX. Sometimes it requires HW/HX.

4.2.3 In-situ Tests

Along drilling survey, Standard Penetration Test (SPT) should be done at every 1.0 meters. Requirements for SPT is described in 4.2.1.

4.2.4 Sampling

Excluding depths of SPT, core sampling should be performed for all depths.

The disturbed samples obtained through SPT should be stored in plastic bags or other air-tight containers so that water contents of the samples are kept unchanged.

Undisturbed samples should be collected in an additional borehole besides the borehole for SPT and core sampling. Undisturbed samples should be collected at each soft soil layers. For soft soil layer of a thickness of 3 meters or more, undisturbed samples should be collected at every 3 meters.

4.2.5 Drilling Logs

Core samples shall be obtained to grasp and record subsurface conditions, such as hardness of soil and rock, weathering conditions, conditions of cracks and filling, groundwater level, situation of spring water, color of water, etc. Such observations of core samples shall be organized and arranged into borehole logs by engineering geologists or geotechnical engineers.

A borehole log shall have the information explained in this section below and be organized in a manner shown in Figure 4-2 and Figure 4-3.

Header of a borehole log shall contain the following information:

a. Name of project
b. Name of borehole (or borehole number)
c. Horizontal coordinate (longitude and latitude in geographic coordinates)
d. Elevation from the sea level
e. Station of the road and distance from the center line
f. Name of drilling company
g. Name of driller

h. Name of geologist or engineer in charge of drilling
i. Type of drilling rig
j. Commencement date of drilling operation
k. Completion date of drilling operation
l. Direction of borehole (vertical or inclined, deg. from vert.)
m. Total depth of borehole
n. Depth (or elevation) of groundwater

Each stratum in a borehole shall be logged. Classification of soil shall conform to ASTM D 2487 or ASTM D 2488 or equivalents. Description of strata shall contain the following items:

a) Soil/rock type or lithology
b) Hardness, relative density or consistency
c) Composition of soil or rock
d) (For gravelly soil) Variation of particle size of gravels and its maximum size
e) (For granular soil) Dominant soil and mixed materials
f) (For fine soil) Homogeneity or mixed materials
g) (For rock) Degree of weathering
h) Texture
i) Structure
j) Color
k) Solution and void conditions
l) Swelling properties
m) Slaking properties
n) (For rock) Intervals of discontinuity (or crack, fissure)
o) (For rock) Angle of discontinuity (or crack, fissure)
p) (For rock) Angle of bedding or foliation
q) Water inflow, loss of circulating water, and heads of confined water encountered during drilling.
r) Additional description such as mineralization, inclusions, vein, and fossils

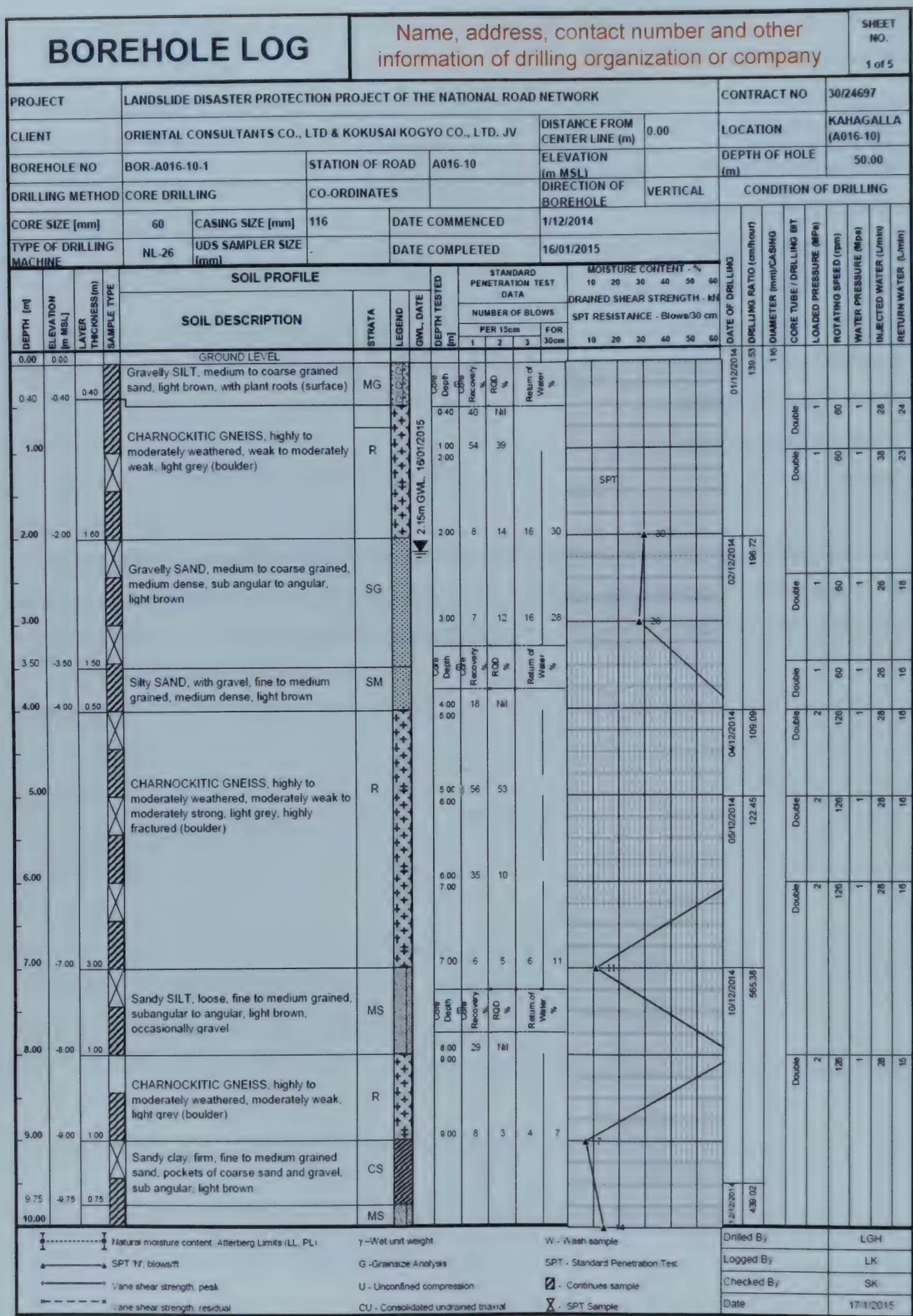
RQD of rock samples obtained in each run shall be measured in a manner explained in Figure 4-4 and recorded in borehole logs.

N-value of SPT and ground water table shall be measured in a manner explained in the following sections and recorded in borehole logs.

Such information shown below shall also be recorded and organized into borehole logs as shown in Figure 4-2 and Figure 4-3.

- Date of drilling
- Drilling ratio (m/day or cm/hour)
- Diameter of bore hole
- Depth and diameter of casing
- Type of core tube/drilling bit
- Loaded pressure (MPa)
- Rotating speed (rpm)
- Water pressure (Mpa)
- Injected water (l/min)
- Return water (l/min)

Core samples shall be properly stored in core boxes as shown in Figure 4-5. Core boxes shall have markings to identify the core samples stored inside. After stored in core boxes, core samples shall be photographed in a manner shown in Figure 4-5.



BOREHOLE LOG

Name, address, contact number and other information of drilling organization or company

SHEET NO. 1 of 5

PROJECT	LANDSLIDE DISASTER PROTECTION PROJECT OF THE NATIONAL ROAD NETWORK			CONTRACT NO	30/24697
CLIENT	ORIENTAL CONSULTANTS CO., LTD & KOKUSAI KOGYO CO., LTD. JV	DISTANCE FROM CENTER LINE (m)	0.00	LOCATION	KAHAGALLA (A016-10)
BOREHOLE NO	BOR-A016-10-1	STATION OF ROAD	A016-10	ELEVATION (m MSL)	
DEPTH OF HOLE (m)	50.00				
DRILLING METHOD	CORE DRILLING	CO-ORDINATES		DIRECTION OF BOREHOLE	VERTICAL
CORE SIZE [mm]	60	CASING SIZE [mm]	116	DATE COMMENCED	1/12/2014
TYPE OF DRILLING MACHINE	NL-26	UDS SAMPLER SIZE [mm]	-	DATE COMPLETED	16/01/2015

SOIL PROFILE

DEPTH [m]	ELEVATION [m MSL]	LAYER THICKNESS (m)	SOIL DESCRIPTION	STRATA	GWL, DATE
0.00	0.00		GROUND LEVEL		
0.40	-0.40	0.40	Gravelly SILT, medium to coarse grained sand, light brown, with plant roots (surface)	MG	2.15m GWL, 16/01/2015
2.00	-2.00	1.60	CHARNOCKITIC GNEISS, highly to moderately weathered, weak to moderately weak, light grey (boulder)	R	
3.50	-3.50	1.50	Gravelly SAND, medium to coarse grained, medium dense, sub angular to angular, light brown	SG	
4.00	-4.00	0.50	Silty SAND, with gravel, fine to medium grained, medium dense, light brown	SM	
7.00	-7.00	3.00	CHARNOCKITIC GNEISS, highly to moderately weathered, moderately weak to moderately strong, light grey, highly fractured (boulder)	R	
8.00	-8.00	1.00	Sandy SILT, loose, fine to medium grained, subangular to angular, light brown, occasionally gravel	MS	
9.00	-9.00	1.00	CHARNOCKITIC GNEISS, highly to moderately weathered, moderately weak, light grey (boulder)	R	
9.75	-9.75	0.75	Sandy clay, firm, fine to medium grained sand, pockets of coarse sand and gravel, sub angular, light brown	CS	
10.00				MS	

DEPTH TESTED [m]	Core Depth m	Core Recovery %	RQD %	Return of Water %	SPT Resistance (Blows/30 cm)
0.40	40	Nil			
1.00	54	39			
2.00	8	14	16	30	30
3.00	7	12	16	28	28
4.00	18	Nil			
5.00	56	53			
6.00	35	10			
7.00	6	5	6	11	11
8.00	29	Nil			
9.00	8	3	4	7	7

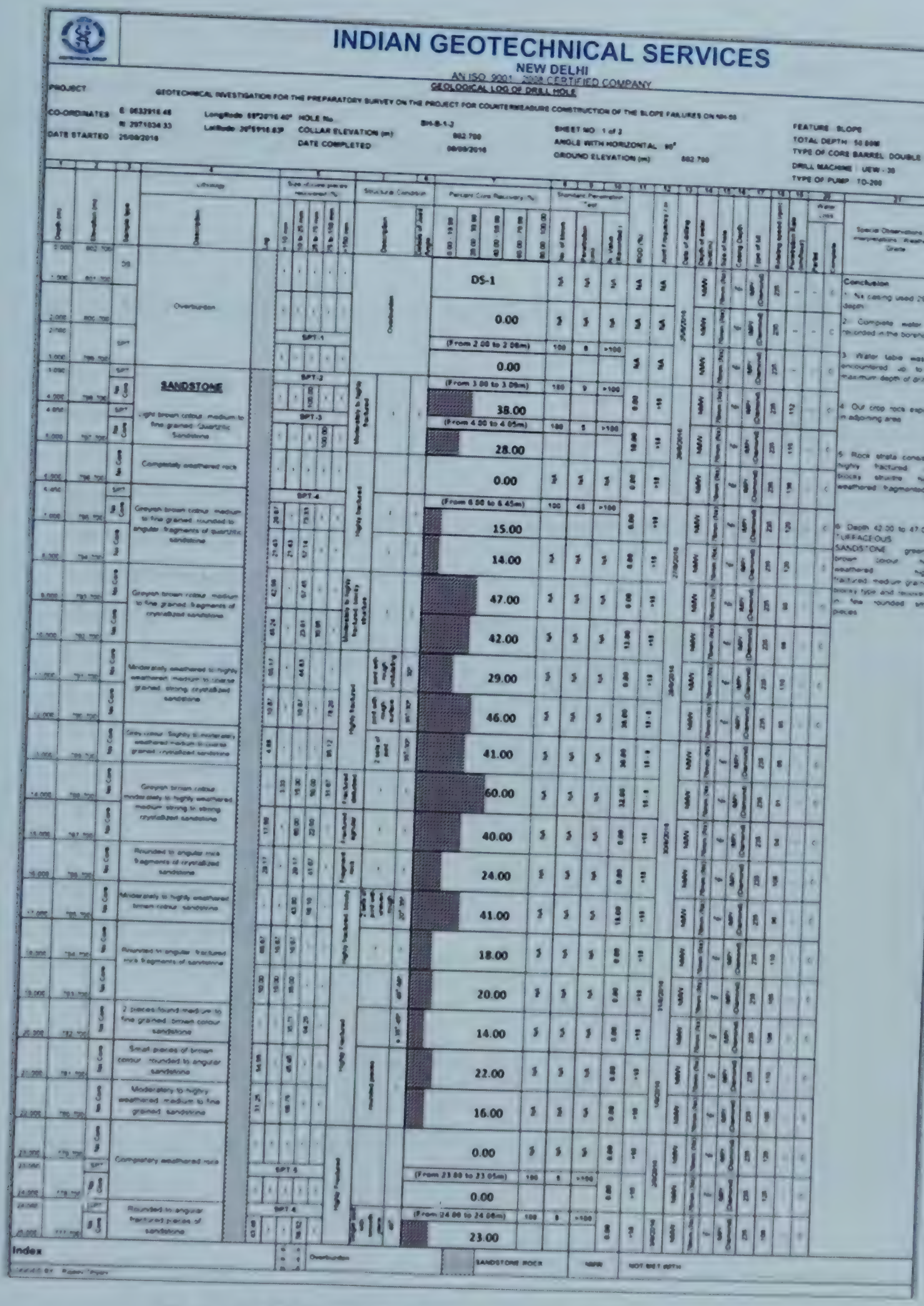
DATE OF DRILLING	DRILLING RATIO (cm/hour)	DIAMETER (mm)/CASING	CORE TUBE / DRILLING BIT	LOADED PRESSURE (MPa)	ROTATING SPEED (rpm)	WATER PRESSURE (Mpa)	INJECTED WATER (L/min)	RETURN WATER (L/min)
01/12/2014	139.53	116						
			Double	1	60	1	28	24
			Double	1	60	1	38	23
02/12/2014	196.72							
			Double	1	60	1	26	18
			Double	1	60	1	26	16
04/12/2014	109.09		Double	2	126	1	28	16
05/12/2014	122.45		Double	2	126	1	28	16
			Double	2	126	1	28	16
10/12/2014	565.38							
			Double	2	126	1	28	15
12/12/2014	439.02							

Natural moisture content, Atterberg Limits (LL, PL)
SPT 'N', blows/ft
Vane shear strength, peak
Vane shear strength, residual
γ – Wet unit weight
G - Grainsize Analysis
U - Unconfined compression
CU - Consolidated undrained triaxial
W - Wash sample
SPT - Standard Penetration Test
- Continues sample
- SPT Sample

Drilled By	LGH
Logged By	LK
Checked By	SK
Date	17/1/2015

Figure 4-2 Drilling Log / Borehole Log (Japanese Style)

Source: JICA Project Team



INDIAN GEOTECHNICAL SERVICES

NEW DELHI

AN ISO 9001 : 2008 CERTIFIED COMPANY

GEOLOGICAL LOG OF DRILL HOLE

PROJECT: GEOTECHNICAL INVESTIGATION FOR THE PREPARATORY SURVEY ON THE PROJECT FOR COUNTERMEASURE CONSTRUCTION OF THE SLOPE FAILURES ON NH-55

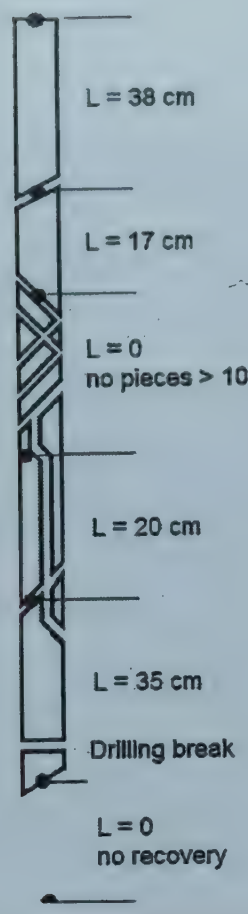
CO-ORDINATES: E 0632916.48 N 2971034.33 | Longitude 88°20'16.40" Latitude 26°59'16.63" | HOLE No. BH-B-1-2 | COLLAR ELEVATION (m): 982.700 | DATE COMPLETED: 06/09/2016 | DATE STARTED: 25/08/2016 | SHEET NO: 1 of 2 | ANGLE WITH HORIZONTAL: 90° | GROUND ELEVATION (m): 982.700

FEATURE: SLOPE | TOTAL DEPTH: 50.00M | TYPE OF CORE BARREL: DOUBLE TUBE | DRILL MACHINE: UEW - 30 | TYPE OF PUMP: TO-200

Index

Figure 4-3 Drilling Log / Borehole Log (Example in India)

Japan International Cooperatino Agency, "Preparatory Survey on the Project for Counter measure Construction for the Slope Failures on National Highway 55," 2017



Total length of core run = 200 cms

$$RQD = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20 + }{200} \times 100 = 55\ \%$$

Figure 4-4 Procedure for measurement and calculation of RQD (After Deere, 1989)

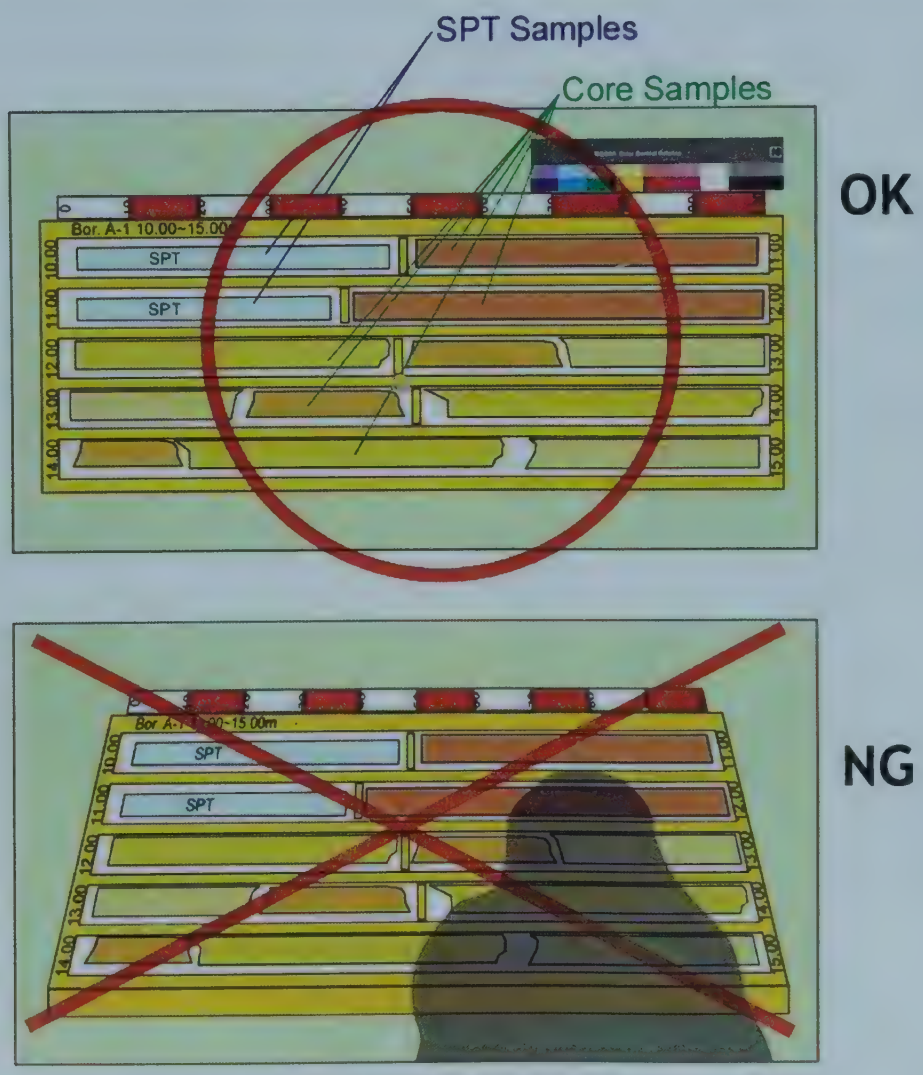


Figure 4-5 Proper manner of Taking Core Photographs

Source: JICA Project Team

4.2.6 Groundwater Level

Groundwater level in a borehole shall be measures before and after each day of work. Measured groundwater levels shall be dully recorded in drilling daily records so that it can be used for groundwater analysis described in Chapter 1.1.2(2) of the Slope Guideline.

4.2.7 Daily Record

The details of drilling shall be recorded for each run:

- Time of drilling
- Meter drilled
- Borehole depth after each run
- Obtained core length
- Name of rock/soil of core samples collected at each run
- Color of core samples
- Diameter of borehole of each run
- Casing type/diameter installed after each run
- Core tube used for core sampling
- Drilling bit used for drilling
- Injected water quantity per minutes (l/min)
- Returned water quantity per minutes (l/min)
- Water pressure (Mpa)
- Rotating speed (rpm)
- Loaded drilling pressure (MPa)
- Depth, length, and blows of SPT, if any
- Depth, length, and sampler of undisturbed sampling, if any.

In addition, the following information shall be recorded for each day of drilling:

- Project name, borehole number, location of borehole, and the date of drilling
- Total drilling time and total length of drilling
- Depth as of the end of the work and total core length obtained in the day
- Groundwater level before the work, with the time measured
- Groundwater level after the work, with the time measured
- Signatures of the operator in charge and the supervisor at the site.

Daily Drilling Operation Report

Project Name: **Landslide Protection Project**

Borehole Number: **Bor - 015 - 3**

Date: **15 May 2016**

Location: **Station Km 14**

TIME [hh:mm]		METERAGE [GL-m]			BOREHOLE DEPTH [GL-m]	CORE LENGTH [m]	ROCK NAME [Soil / Highly Weathered / Weathered / Bolder / Rock]	COLOR [Sample and/or Water]	BOREHOLE SIZE [mm]	CASING TYPE [mm]	CORE TUBE	DRILLING BIT	INJECTED WATER [L/min]	RETURNED WATER [L/min]	WATER PRESSURE [kPa]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kPa)	SPT : Standard Penetration Test (DISTURBED SAMPLE)				UNDISTURBED SAMPLE			
From	To	From	To	Meter drilled																					
8.30	13.05	9.00	11.20	2.20	Reamed & reset NW Casing 9.00 to 11.20m				92.00	88.00	Diamand casing bit														
13.05	13.20	11.20	11.60	0.40	11.60	_	Sandy CLAY	Whitish grey	76.00	-	Single	Metal	0.00	0.00	0.00	65.00	1.00	-	-	-	-	-	-	-	-
13.50	14.10	11.60	11.93	0.33	11.93	_	Sandy SILT	White	76.00	-	-	-	-	-	-	-	-	11.60	11.93	0.33	>50	-	-	-	-
14.30	14.45	11.93	12.40	0.47	12.40	_	Sandy SILT	White	76.00	-	Single	Metal	0.00	0.00	0.00	65.00	1.00	-	-	-	-	-	-	-	-
14.50	15.25	12.40	13.30	0.90	13.30	0.64	Khondalite	Brown	76.00	-	Double	Diamond	16.60	10.00	2.00	65.00	2.00	-	-	-	-	-	-	-	-
15.50	17.30	13.30	14.80	1.50	14.80	0.62	Khondalite	Brown	76.00	-	Double	Diamond	25.00	8.00	1.00	150.00	2.00	-	-	-	-	-	-	-	-

Total Drilling Time [hh:mm]	Total Matege [m/day]	Total Depth [m]	Total Core Length [m]	Ground Water Level Before Work	Measured time	Ground Water Level After Work	Measured time
1.45	3.60	14.80	1.26	6.70	8.30	6.90	16.30

Site In Charge (Geologist/Engineeer)

.....

Client Approval

.....

Figure 4-6 Daily Record of Drilling Survey

Source: JICA Project Team

Figure 4-6 shows an example of daily record of drilling survey.

4.2.8 Use of borehole for monitoring

After the drilling, borehole shall be used for monitoring, depending on requirements. Preparing monitoring well with a borehole and observation of groundwater level will provide basic information for slope stability analysis.

If a borehole is planned outside of shoulder of high cut slope, such boreholes may remain after the cutting. For such case, installing monitoring equipment for underground displacement and performing periodic observation may provide essential information of slope stability after completion of cutting.

The similar case is supposed for high filling. A borehole planned outside of toe of high filling may remain after the filling work. Installing monitoring equipment for underground displacement and performing periodic observation may provide essential information of stability of high embankment after completion of filling.

5 Laboratory Tests (Normal Case)

Basic property of soil can be obtained by physical lab tests using disturbed samples of SPT. Undisturbed samples can be used for compression and consolidation settlement tests, which is essential for soft soil improvement.

Basic property of rock can be obtained by physical and mechanical lab tests using core samples. The strength obtained from core samples reflects the strength of material. The strength of rock mass is often weaker than the material due to cracks, joints, bedding, fractured zones, and other weak planes included in rock mass.

6 Geophysical Exploration (Normal Case)

6.1 Seismic Exploration

For high and long cut section, seismic exploration may be planned to obtain 2-D image of seismic speed distribution, which may imply hardness or density of cracks of bed rock.

Survey lines for a high and long cut section may be planned as follows:

- Main survey line is on the planned center line of highway, covering whole stretch of the high and long cut.
- Sub survey lines crossing the main survey line with right angle are to cover the whole cut section at the crossing and some marginal length beyond the shoulder of the planned cutting.

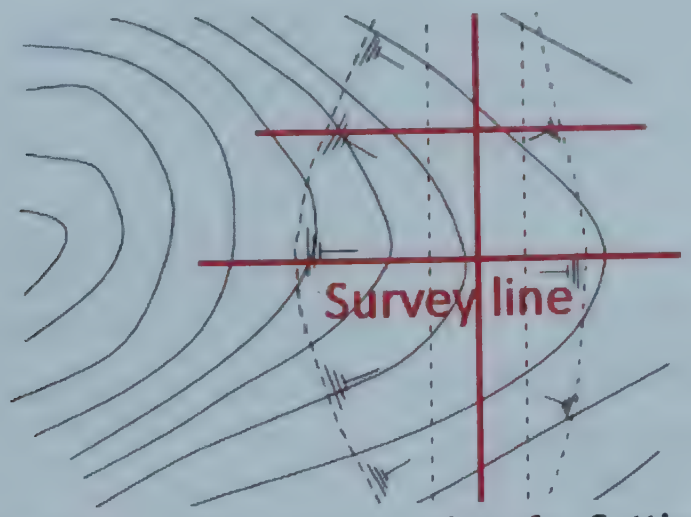


Figure 6-1 Plan of Survey Lines for Cutting

Source: JICA Project Team

- To achieve deeper exploration depth, remote shot points located at certain distance from both ends of the survey line is effective and thus shall be so planned.
- With explosives, seismic exploration with refraction method may explore up to 100 ~ 200 meters below ground, depending on the length of survey lines and arrangements of shot points. In case explosives are not permitted, hitting plate method may substitute, with stacking method to magnify the energy of the shots.
- Geophones to pick up seismic wave are usually arranged at every 5 ~ 10 meters intervals on the survey lines.
- Shot points are arranged at every 5 ~ 10 meters intervals on the survey lines for high density exploration, which enable geo-tomographic analysis of elastic wave distribution on 2D images.

7 Topographic Mapping and Geological Field Survey (Special Case)

7.1 Photo or Map Reading

In case photo or map reading was not conducted for the section where landslides or gravity deformation disturbs or is to disturb the hill roads, to verify the necessary areas for

mapping and surveys, photo or map reading should be done before planning topographic mapping and other surveys. Photo or map reading should be done along with field surveys to well understand the targeted landslide or gravity deformation.

7.2 Purpose

The purpose of Topographic mapping and geological field survey for Special Case is to provide basic information for plan and design of countermeasures / mitigation measures against landslide and/or gravity deformation.

A block of landslide / gravity deformation often gets activated in conjunction with the surrounding blocks; activity of a block of landslide often affects surrounding dormant landslides and/or gravity deformation blocks. In addition, movement of a landslide removes supports of adjacent steep slopes and then destabilizes them.

Thus the area for topographic mapping and geological field survey should encompass the surrounding areas where dormant landslides and/or gravity deformation blocks lie around the landslide/gravity deformation actually affecting the hill roads.

7.3 Requirements

Other than the requirements shown in 3.3 of this document, profiles of landslide/gravity deformation blocks should be surveyed. If the size of such blocks are large, in addition to profiles, cross section surveys of landslide/gravity deformation blocks should be added.

Basically, survey line for main profile of a landslide block should be set on the center of such block with the direction parallel to the movement of landslide. Chapter 1.1.6 of the Slope Guideline should be referred for setting survey lines for landslides with landslide clay.

If the targeted slope is steep, main profile should be parallel to the steepest gradient of slope. If the steepest gradient changes its course in the middle of slope, survey line for profile should also be changed.

On the profiles or cross sections of the blocks of landslide/gravity deformation, boreholes of drilling survey are often planned. The actual locations of such boreholes should be indicated on site with survey stakes or other appropriate methods, to lead drilling operator to the proper locations. Borehole locations should be shown on profiles and cross sections

as well, to facilitate preparation of landslide profile for slope stability analysis.

Topographic and geological map should show the boundaries of the blocks of landslide/gravity deformation with the topographic/geological features indicating the blocks.

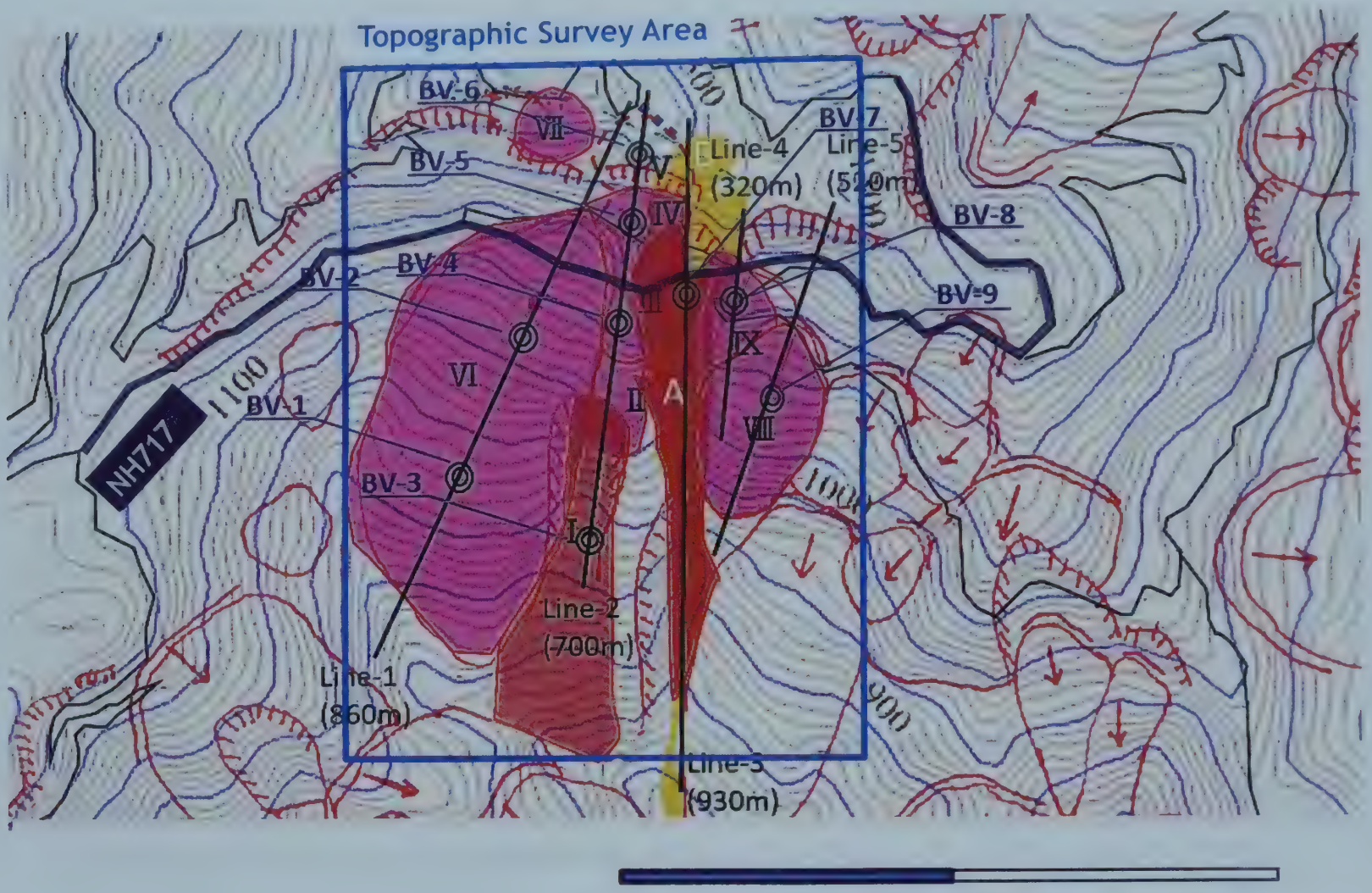


Figure 7-1 Example of Topographic Survey Area and Survey Lines of Profiles of Blocks

Source: JICA Project Team

8 Drilling Surveys and Laboratory Tests along with Related Surveys (Special Case)

In addition to the requirements in 4.2 in this document, drilling survey should be done on the profiles/cross sections of the blocks of landslide/gravity deformation.

8.1 Borehole Locations

For smaller blocks, a borehole should be placed on the profile where shape of slip surface/loosened zone can be well understood; usually a borehole is planned in the middle

of the block. If a landslide block is divided into multiple sections by scarps, which suggests tension cracks of multiple slip surface, boreholes should be so planned at each divided section that shapes of multiple slip surface could be understood.

For larger blocks, multiple boreholes should be so planned on the main profile that multiple slip surface could be understood. Boreholes may be planned on sub profiles or cross sections, if necessary.

For multiple block landslides, at least one borehole should be planned at each block.

Figure 7-1 shows an example of allocating boreholes at multiple blocks of gravity deformation; boreholes are shown by symbols of two concentric circles.

8.2 Depth of Borehole

The depths of borehole should so planned that base rock under the deepest slip surface could be confirmed by drilling. For a reference of planning the depths of boreholes, Chapter 1.2.4 of the Slope Guideline provides statistical suggestion to the depth of slip surface, which can be applied to landslide with landslide clay on slip surface.

On the other hand, for deciding necessary depths of boreholes at blocks of gravity deformation, appearance of neighboring outcrop may provide suggestions. At the site of deep-seated landslides, loosened condition of bedrock are often observed, based on which thickness of adjacent blocks of gravity deformation is estimated.

8.3 Groundwater Observation during Drilling

As explained in 4.2.6 in this document, measuring groundwater level in boreholes in the morning and evening of each working day before and after the work is essential to understand groundwater condition through Groundwater Variation Analysis Chart described in Chapter 1.1.2(2) of the Slope Guideline.

8.4 Monitoring

As much as possible, boreholes after drilling should be used for landslide monitoring such as show below:

- Underground displacement
  - Borehole inclinometer

- Pipe strain gauge
- Borehole extensometer
- Short pipe test

◆ Groundwater monitoring

- Monitoring well for manual observation
- Monitoring well for automatic observation with groundwater level meter.
- Monitoring well for piezometer to measure pore water pressure at slip surface

Basically, landslide monitoring requires dedicated borehole for each method; monitoring well cannot be used for borehole inclinometer. So if monitoring for underground displacement and groundwater monitoring is planned at the same location, two boreholes should be drilled.

Borehole extensometer, however, can be attached to PVC pipes for monitoring well. So additional bore hole is not required. So is short pipe test, which can utilize PVC pipe of monitoring well.

8.5 Borehole imagery

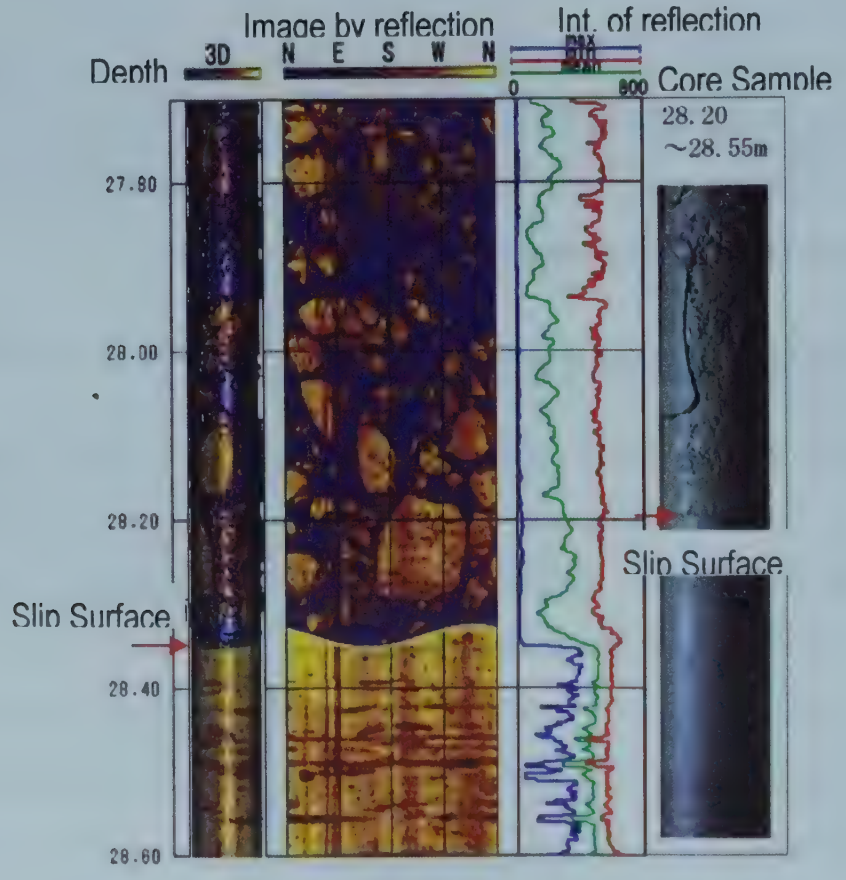


Figure 8-1 Detection of Slip Surface with BHTV

Source: JICA Project Team

As understood from Chapter 1.2.4 of the Slope Guideline, core samples obtained through drilling survey provide essential and valuable information for deciding depths of slip surfaces of landslides with landslide clay and loosened zones of gravity deformation as well as boundary between debris and bedrock or weathered/cracked/fractured zones and fresh/intact bedrock. Obtaining high quality core samples with sufficient core recovery ratio of such slip surfaces and loosened zones as well as debris with boulders and cracked/fractured zones of rock is often difficult unless proper and well-maintained tools are used and accurate and meticulous manner for obtaining core sample is performed during drilling, which are yet well widespread in the hilly regions.

In case high quality core samples with sufficient quality is not provided from drilling, other ways to understand subsurface conditions should be considered. Borehole imagery is one of such methods to supplement information from core samples from drilling and explained in Chapter 1.1.2(2) (iv) in the Slope Guideline. Borehole imagery survey was developed in the energy sector for exploiting fossil fuel. It is applicable for landslide engineering as well.

9 Geophysical Exploration (Special Case)

Geophysical exploration is also useful in Special Case when it is applied to a survey for blocks of gravity deformation. Gravity deformation is featured by creep deformation of loosened zone of bedrock, where cracks and fissures are densely developed. Among such cracks and fissures, groundwater may be stored as fissure water which should affect resistivity of the ground. Thus resistivity survey with geo-tomography is worth to perform to understand distribution and depths of loosened zones of gravity deformation, which may supplement results of core boring.

Changes in groundwater storage are detected by sudden changes in rates of variation of observed apparent resistivity of the ground. Chapter 1.3.4(3) shows the example of such sudden changes in observed apparent resistivity.

Resistivity survey with geo-tomography should be performed on profile of blocks of landslide / gravity deformation. Figure 7-1 shows an example of arranging profiles of multiple blocks of gravity deformation.

10 Instrumental Monitoring (Special Case)

Instrumental monitoring is also featured in Chapter 6 of IRC: SP: 106 "Engineering Guidelines on Landslide Mitigation Measures for India Roads" and in Chapter 9 of IRC Special Report No.15: "State of the Art: Landslide Correction Techniques".

Instrumental monitoring should be so planned to grasp movement of landslide/gravity deformation as well as groundwater level/pore water pressure and corresponding rainfall.

- Underground displacement
  - Extensometer
  - Ground inclinometer
  - Crack gauge
  - Displacement measurement with batter board (finishing) stakes
- Underground displacement
  - Borehole inclinometer
  - Pipe strain gauge
  - Borehole extensometer
- Groundwater monitoring
  - Monitoring well for manual observation
  - Monitoring well for automatic observation with groundwater level meter.
  - Monitoring well for piezometer to measure pore water pressure at slip surface
- Rainfall observation
  - Automated rain gauge (tipping bucket or other methods)
  - Manual rain gauge

As described in Chapter 1.1.5 and 1.1.6(3) of the Slope Guideline, selection of groundwater monitoring is important. To understand pore water pressure on slip surface of landslide, either of the following methods should be employed:

- Piezometer placed on the slip surface
- Dedicated groundwater monitoring well, which has perforation only at the section of slip surface and upper and lower packers to limit the section and to stop inflow of groundwater from outside of the section.

For each method, depth of slip surface should be understood before placing piezometer or dedicated groundwater monitoring well.

To understand general groundwater condition of gravity deformation or slope failure, monitoring well with perforation placed for full depths can be used, unless groundwater at particular aquifer or fissure zone is required. For observation, automated groundwater level gauge is favorable. If it is not available, periodical manual observation, weekly or monthly with additional observation after heavy rainfalls, may substitute.

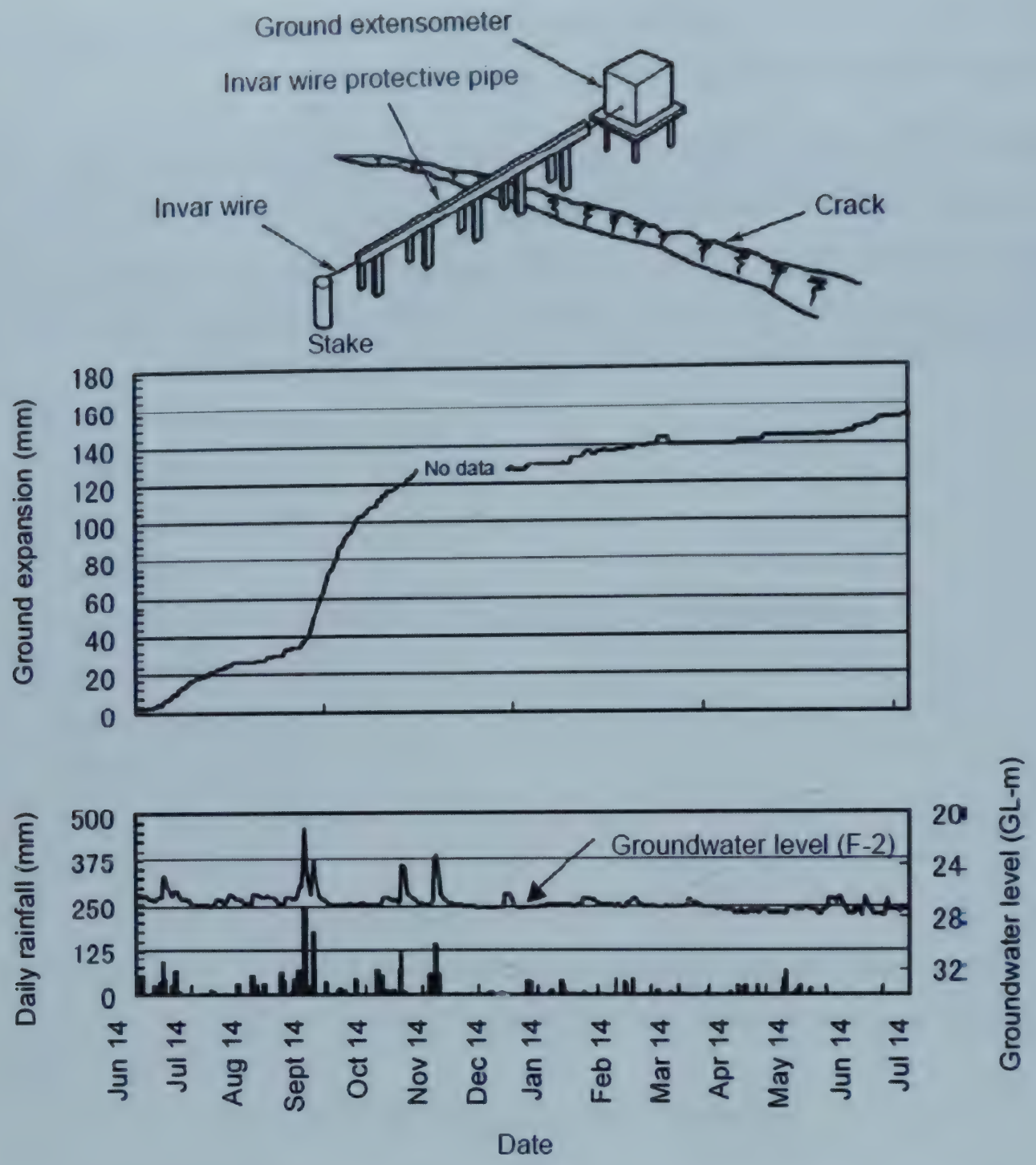


Figure 10-1 Extensometer (Top: Installation, Bottom: Measurement)

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)

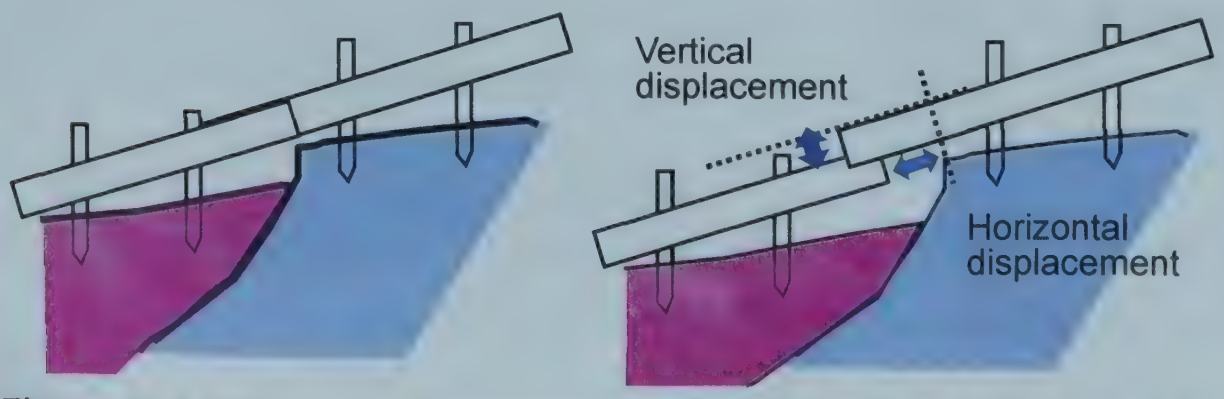


Figure 10-2 Displacement Measurement with Batter Board (Finishing) Stakes

Source: JICA Project Team

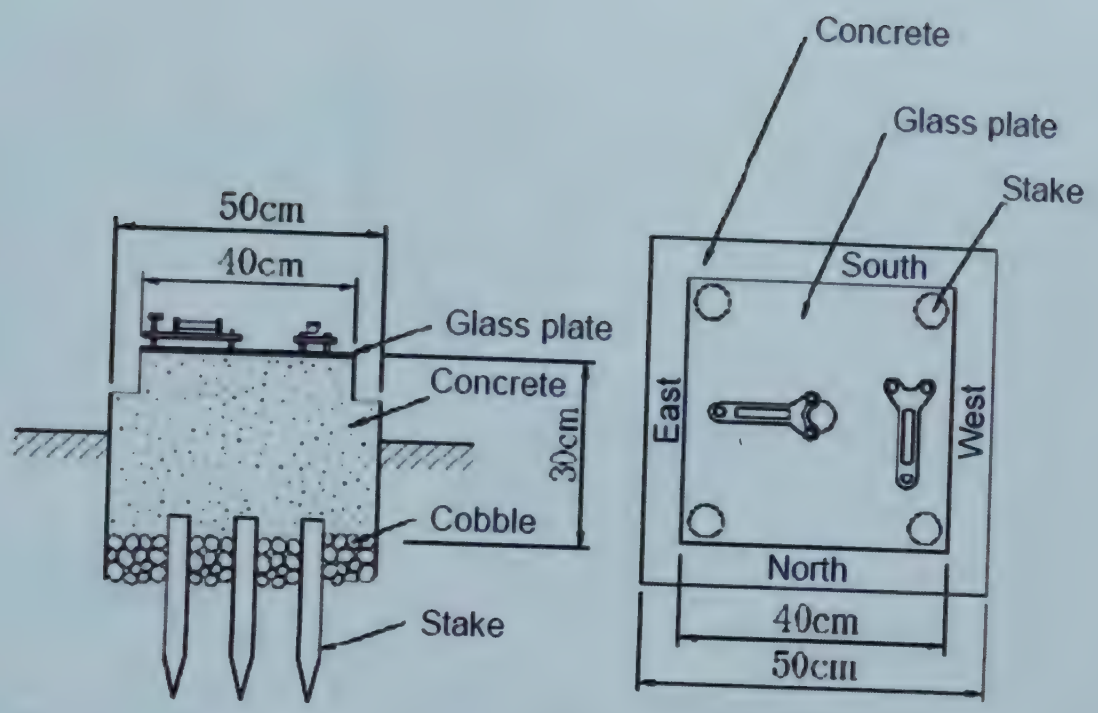


Figure 10-3 Ground Inclinometer

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)

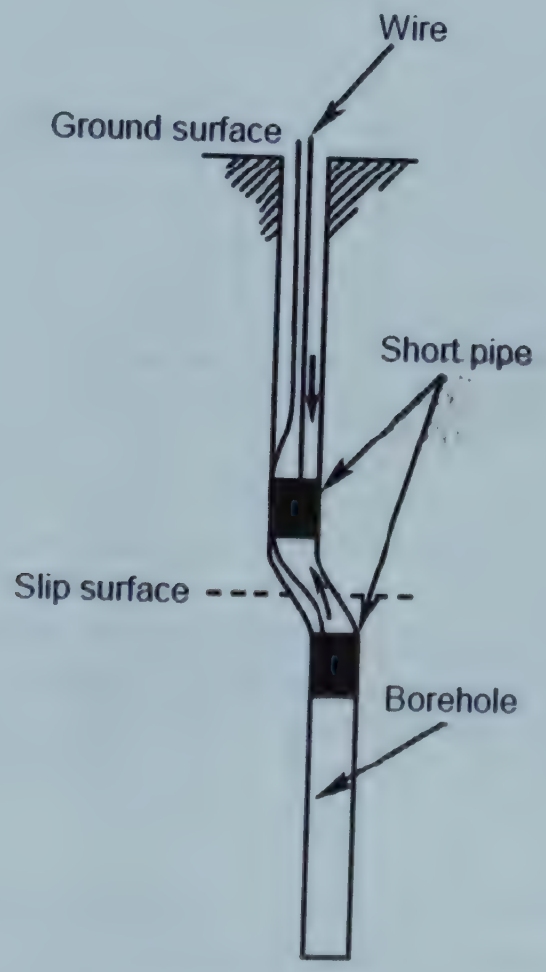
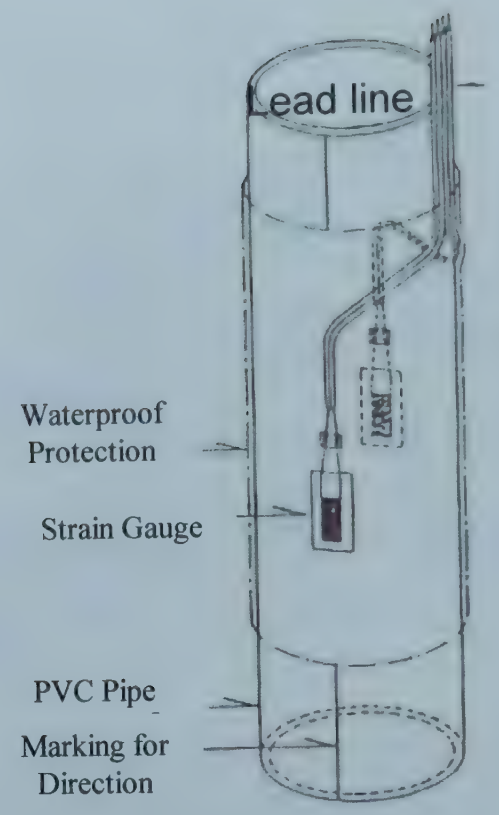


Figure 10-4Evaluation of Slip Surface Using Short Pipe (Short Pipe Test)

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)



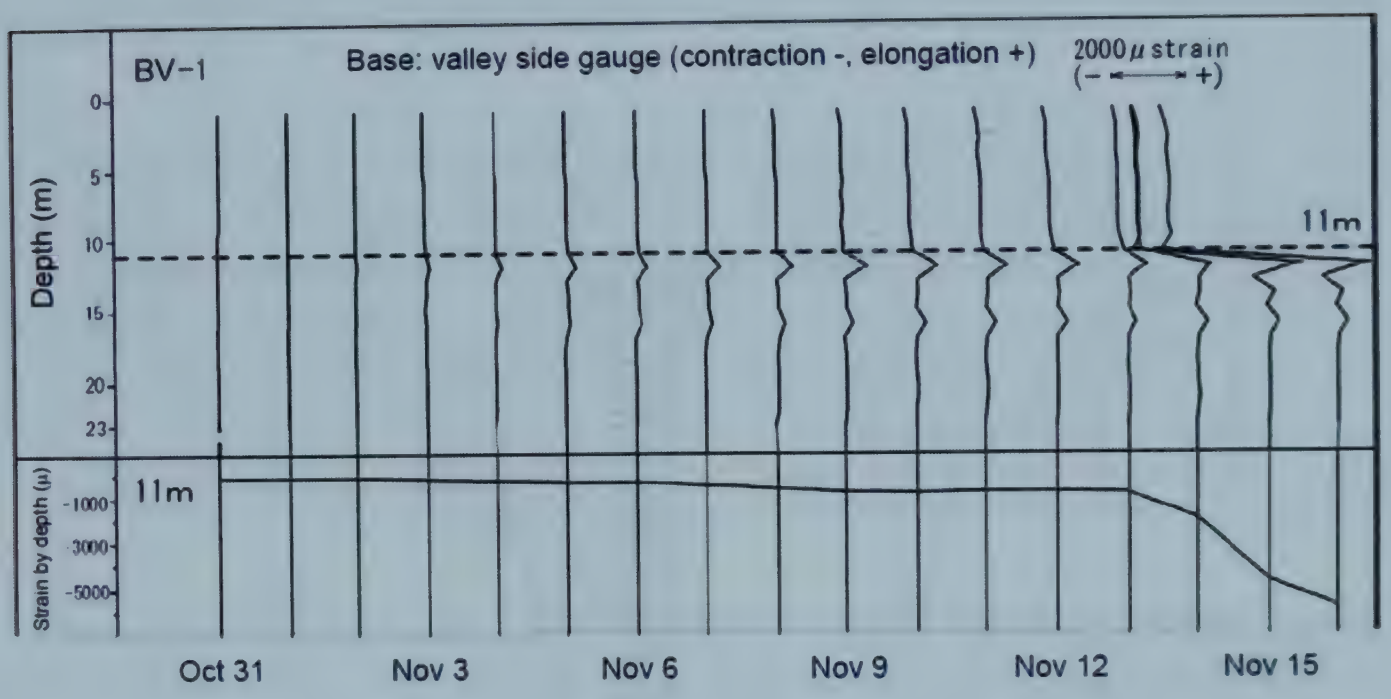
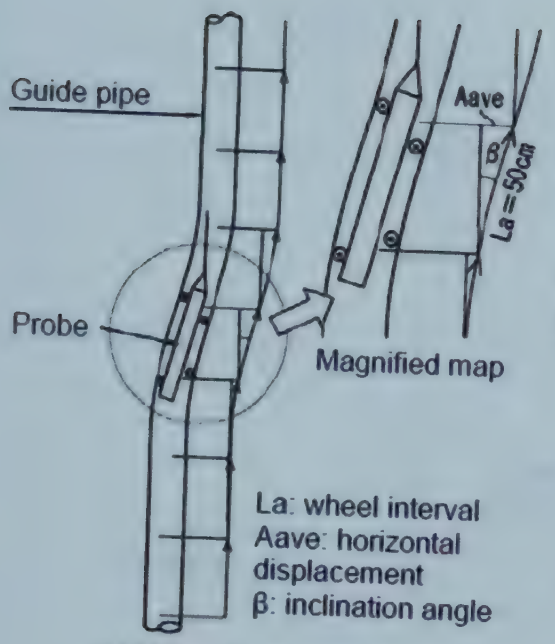
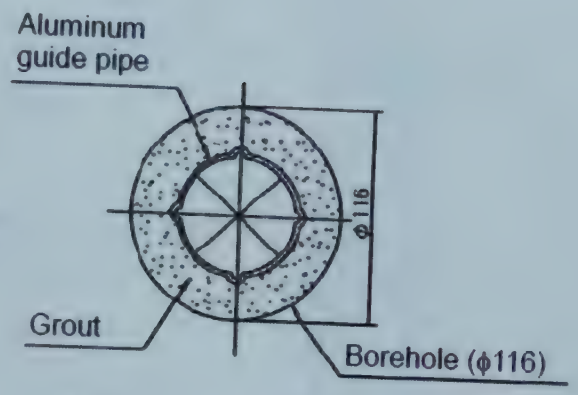


Figure 10-5 Cumulative Strain Obtained by Pipe Strain Gauge

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)



(a) Measuring method using a borehole inclinometer



(b) Cross section of a borehole inclinometer

(c) Guide pipe

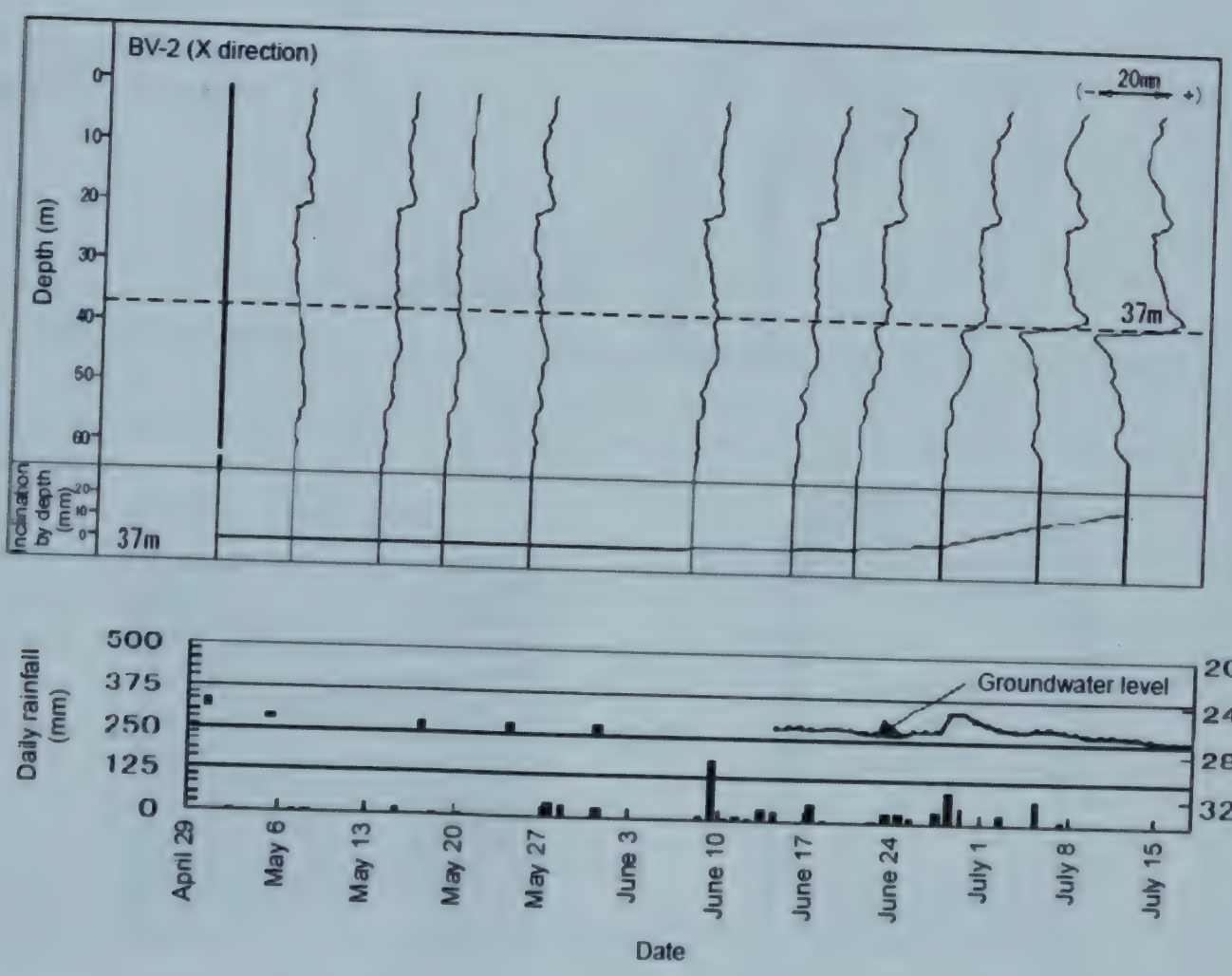


Figure 10-6 Cumulative Inclination Measured by Borehole Inclinometer

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)

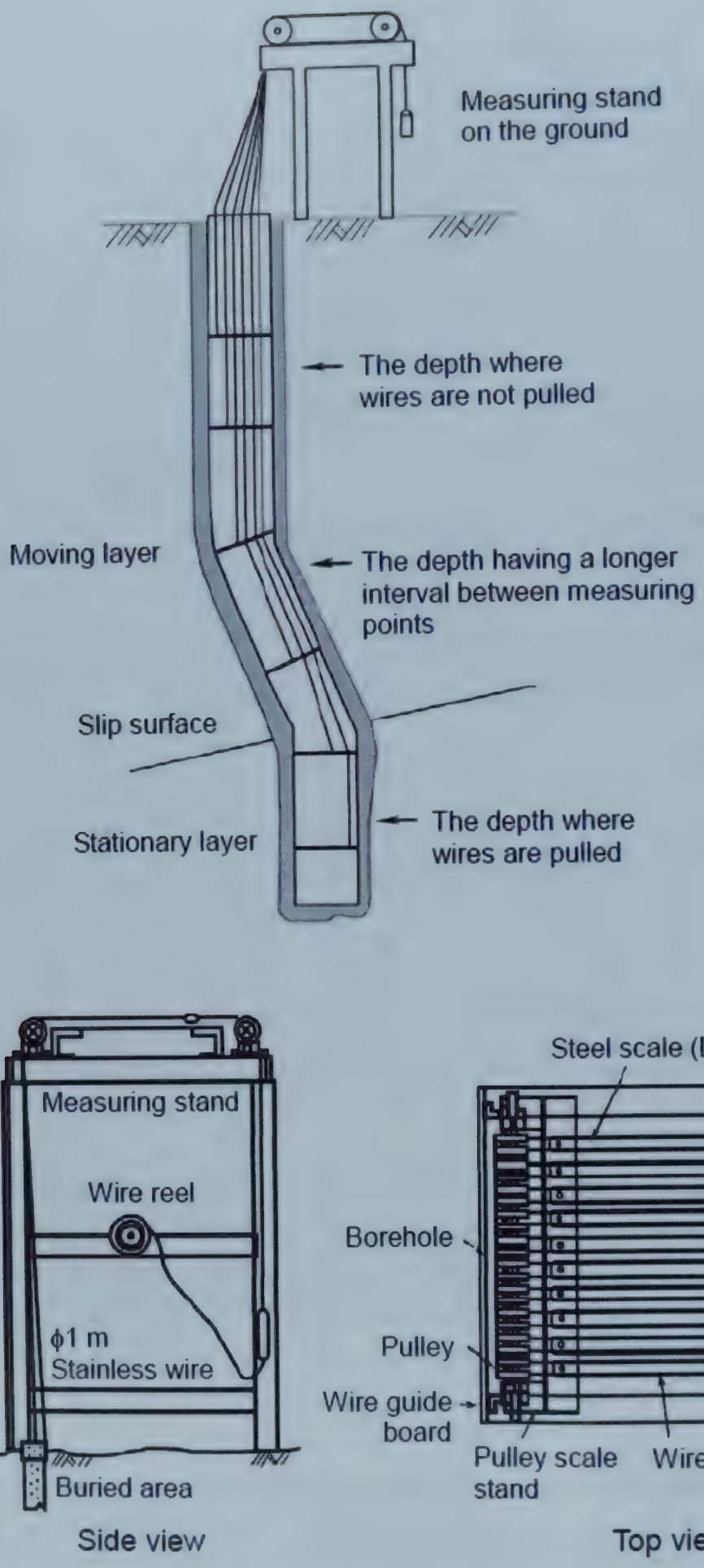


Figure 10-7Borehole Extensometer (Top: Outline, Bottom: Measuring System)

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)

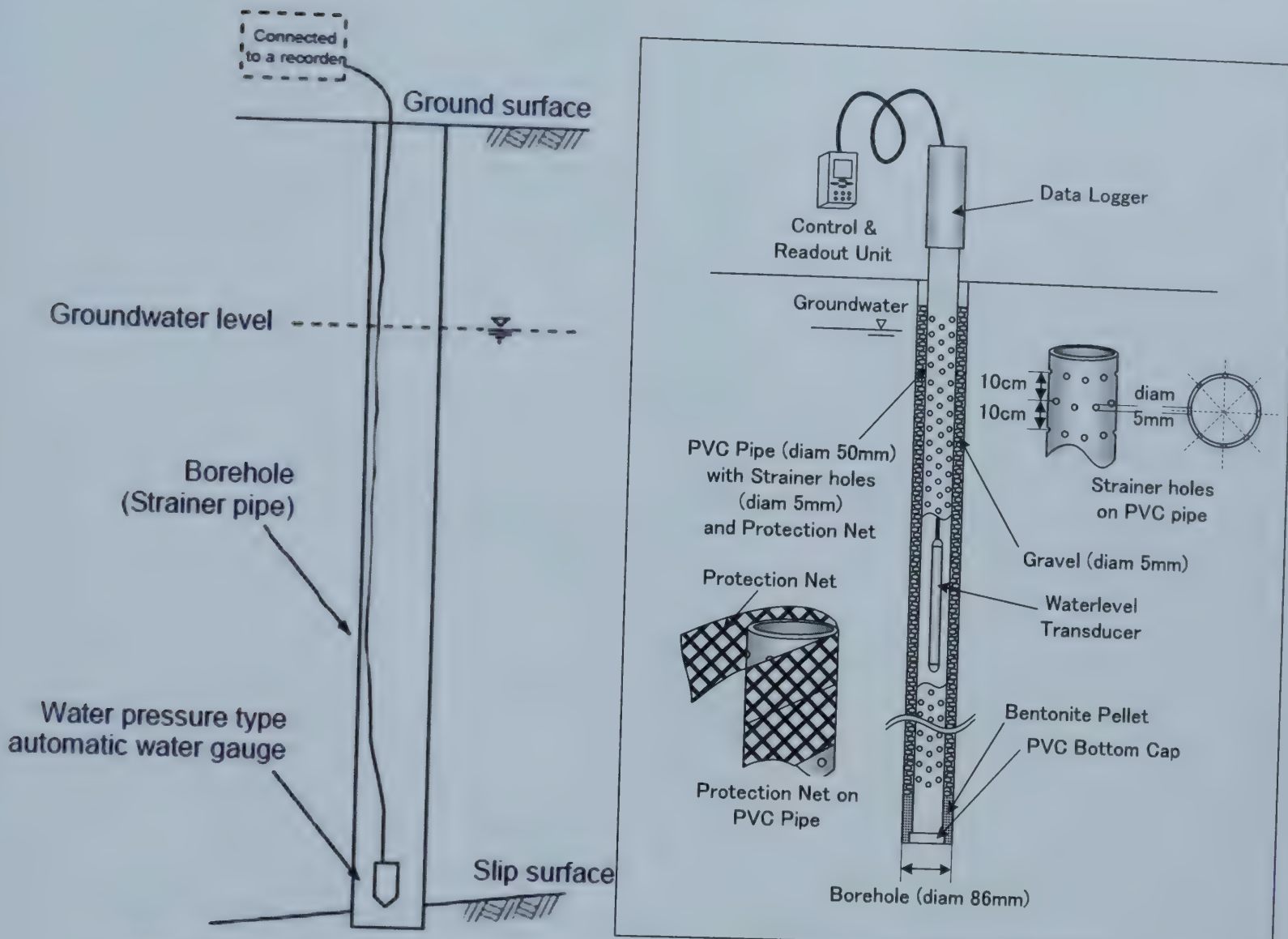
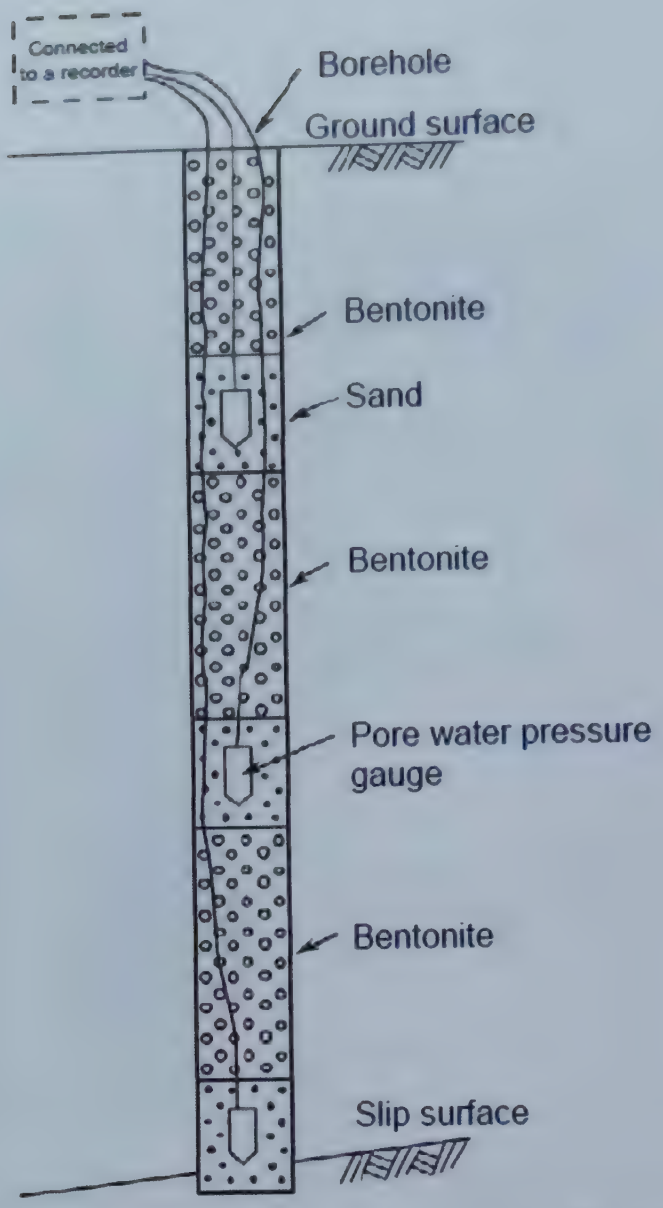


Figure 10-8 Water Pressure Type Automatic Groundwater Level Meter

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)



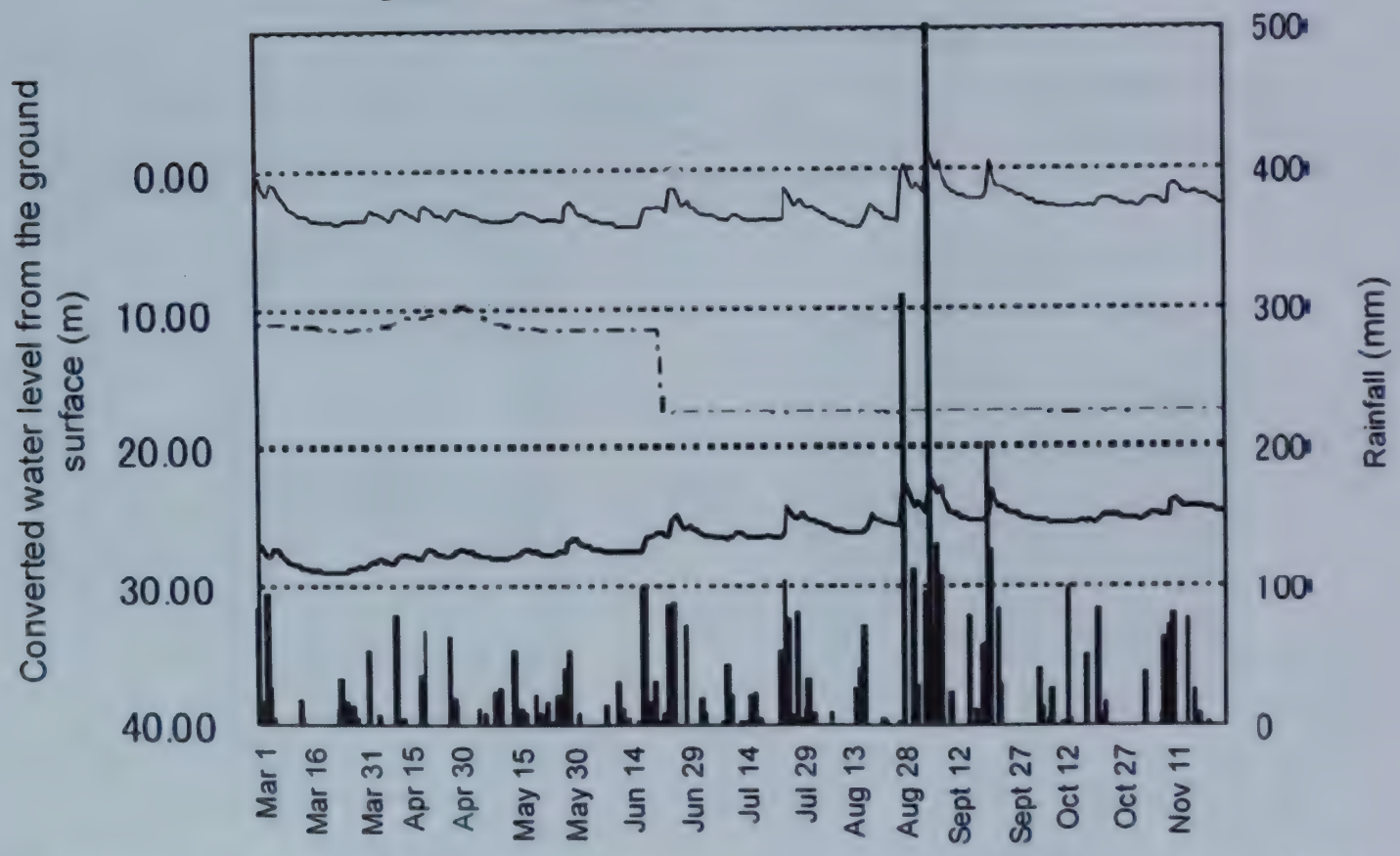


Figure 10-9 Top: Burial Type Automatic Piezometer, Bottom: Measurement of Pore Water Pressure

Source: PWRI Technical Note No. 4077, Guidelines for Landslide Prevention Technologies. September 2007, Landslide Research Team, Erosion and Sediment Control Research Group, Public Works Research Institute (PWRI)

Appendix II

Recommendation on Countermeasure Design: NH10

Recommendations on Design of Countermeasures against Landslides on NH10 in the State of Sikkim

July 2019

JICA Experts for Capacity Development Project on Highways in Mountainous Regions

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1 Introduction

1.1 Recommendations on Design

This report was made to provide the recommendations for the DPR preparation for the pilot project to construct countermeasures against landslides at Km 70 + 932, Km 71 + 010, and Km 71 + 700 on NH10 in the state of Sikkim.

As per the discussions summarized in the Office Memorandum dated on the 13th of December, 2018, and issued by AEE (EAP) of MoRTH, the recommendations aim to provide the main ideas of the design of countermeasures, which are required to stabilize the aforementioned landslides. For such purpose, the recommendations include plan views and typical cross sections of the countermeasures as well as the results of slope stability analyses and calculations for representative countermeasures, such as ground anchor and rock bolts, which are attached to the Appendix.

This report also intends to provide technical assistance to Indian engineers in preparing / updating the DPR based on the recommendations. The guidelines prepared by the Project provide the basic knowledge and procedure for the design. On the other hand, this report provides specific procedures carried out by JICA Experts to prepare the recommendation for the design as well as the practical procedure which should be carried out by Indian engineers to prepare / update the DPR based on the recommendations. These procedures are mainly explained in Chapter 4 of this report.

The draft of the slope guideline is already completed for Chapter1 Investigation, Chapter2 Design, and half of Chapter3 Construction, which were submitted to MoRTH via a letter issued from JICA Experts with a reference No. JXPTHE-023/2019 dated on the 22nd of May, 2019. We would expect the draft guideline would help Indian engineers.

Since it can be done by Indian engineers with the current knowledge and the existing guidelines, recommendations regarding the common measures, such as surface drainage ditches, are omitted, unless special cares are required for such measures.

1.2 Investigation

Countermeasures against landslides should be so designed to prevent / mitigate specific phenomena relating to landslides. For this meaning, investigation and survey of landslide are definitely important for design. Through investigation and survey, engineers can identify and measure the "enemy" to be "neutralized" by countermeasures. The slope guideline prepared by the Project, therefore, paid substantial attentions to the part of

investigation and survey.

For the pilot project site of NH10, the following survey reports were submitted to MoRTH from JICA Experts.

(1) Advisory Report of Site Survey on NH717 in the State of West Bengal and NH10 in the State of Sikkim for execution of pilot project, dated on the 28th of January, 2019.
(2) Advisory Report for Landslide Sites of NH10 in the State of Sikkim for Selecting Site for Pilot Civil Work (31st January - 2nd February) dated on the 25th of February, 2019.

Although the latest field survey conducted in June 2019 for the preparation of the recommendations updates some information about the landslide condition, the site report of (2) provides comprehensive understanding of the landslides to be treated in the Pilot Project. Thus the report of (2) should be referred by the engineers for DPR preparation / updating.

Besides the said 2 reports, the following report provides valuable information for improving the quality of survey results relating to landslide investigation.

(3) Comments on Quality and Manners of Surveys by DPR Consultant for NH10 in the State of Sikkim, with a reference No. JXPTHE-031/2019, dated on the 04th of July, 2019.

The Chapter 2 and Chapter 3 of this report briefly explain the results of investigation, which provide basis for slope stability analyses and reasons for selecting certain types of countermeasures. The results also specify areas and requirements for the said countermeasures.

1.3 Schedule of Pilot Project on NH10

As per the said Office Memorandum and the limitation of the period for the Project, the countermeasure works at the sites of NH10 should be completed by December 2020. This objective requires JICA Experts to submit the recommendation by the middle of July 2019, which should be followed by the activities listed below:

1) Preparation / updating the DPR by the middle of August 2019
2) Approval of DPR by NHIDCL by the end of August 2019.
3) Floating tender by the middle of September 2019.
4) Issuing L/A by the end of November 2019
5) Commencement of construction by the middle of December 2019.

6) Completion of the works by the middle of November 2020.

To meet the schedule, JICA Experts paid sincere efforts to overcome difficulties in preparing the recommendations, some of which are explained in the said report of (3).

Due to insufficient area and accuracy of the plan view and cross sections, JICA Experts utilized satellite data to supplement the missing parts of the survey results. At the same time, slope failures at the site changed some surface of the terrain after the survey. These conditions require re-survey of plan view and cross sections before DPR preparation.

On the other hand, monsoon season at the pilot project site in the state of Sikkim usually lasts until the end of August and thus re-survey may start in September, which delays DPR preparation at least for 2 months.

To avoid such delay, preparing DPR with the drawings currently available may be an option. With this option, some errors are inevitably incorporated in DPR, which may be rectified by the DPR consultant after the Contractor for the works surveys the site at the commencement of the work. As long as the shapes of the slope are not drastically changed, however, design policy of the recommendation remains effective.

1.4 Special Engineer for Landslide Countermeasure Works

Design of countermeasures against landslide requires special knowledge and skills, which are not usually shared by ordinary civil engineers. Thus well-experienced engineers in the field of landslide mitigation and countermeasure works should be involved in preparation / updating of DPR.

2 Slide No.1 at Km 70 + 932 and No. 2 at Km 71 + 010,

2.1 Conditions of Slope and Landslides

The conditions of the site are summarized in the following report:

(2) Advisory Report for Landslide Sites of NH10 in the State of Sikkim for Selecting Site for Pilot Civil Work (31st January - 2nd February) dated on the 25th of February, 2019.

At the slope where Slide No.1 and No.2 lie, no outcrops of bedrock appear on the surface. Some parts of boulders and fallen rocks appear on the surface but the slope is covered by debris or colluvium.

The landslides at Slide No.1 and No.2 can be divided into 7 zones:

1) Upper slide area of Slide No.1, where circular failure occurred,
2) Lower slide area of Slide No.1, where debris from upper ran on and peeled off the surface,
3) Slide area of Slide No.2
4) Gentle slopes adjacent to 1) the upper slide area of No.1 or 3) Slide No.2, where possible slide blocks hide,
5) Road side steep slopes
6) Road side steep slope with possible deformation block
7) Road side steep slope with large boulders and fallen rocks

Figure 2-1 Slide No.1 (Left) and No.2 (Right)

Figure 2-2 Zone 1) Upper Slide Area of Slide No.1 (Left) and Soil Pipes (Right)

At the zone 1) upper slide area of Slide No.1, trace of a circular failure was observed along with soil pipes. The soil pipes are formed by the action of seepage water, the pressure of which often causes surface failures. At the zone 1), due to unloading by the slide, potential slide zones may develop beneath the surface. Thus countermeasures to reinforce the slope are required. At the same time, to counteract the seepage water pressure, horizontal drainage borings are required.

Figure 2-3 Zone 2) Lower Slide Area of Slide No.1 (Left) and 3) Slide Area of Slide No.2

The zone 2) shows planner shape of the slop surface where the surface soil was peeled off. The zone 2) seems to be rather stable but protection against erosion is required.

At the zone 3), the similar unstable condition to the zone 1) are expected. Thus the similar countermeasures are expected.

Figure 2-4 Zone 4) Gentle Slopes of Possible Slide (Left, Possible block is surrounded by yellow line and arrows indicate tree bends) and 5) Road Side Steep Slope (Right)

The zone 4) has two areas; the right side of Slide No.1 and the upper slope of Slide No.2. On the surface of the zone 4) adjacent to Slide No.1, tree bends were observed as shown in Figure 2-4. The zone 4) has a potential of slope failures in near future. Thus countermeasures to reinforce the slope is required. In addition, around the toe of the block adjacent to Slide No.1, groundwater springs was observed. Thus horizontal drainage borings should be applied.

The zone 5) exhibits steep slope of colluvium, which has potential of surface failures and erosion. Thus the surface of the zone 5) should be protected by structures. At the same time, the shoulder of the roadside slope should be rounded to ease loosening of the shoulder.

Figure 2-5 Zone 6) Road Side Steep Slope with Possible Deformation Block (Left) and 7) with Large Boulders and Fallen Rocks (Right)

The zone 6) shows more unstable and loosened condition than the zone 5), which is understood from map reading. Thus reinforcement should be added to the countermeasures for the zone 5).

The zone 7) is covered by large boulders and fallen rocks, which accumulate in an unstable manner. Before installing the similar countermeasures to the zone 6), removal of large boulders and fallen rocks are required.

To clarify the conditions of Slide No.1 and No.2, drilling investigation and cross section survey were requested in the letter issued by JICA Expert with a reference No. JXPTHE-008/2019, dated on the 15th of April, 2019.

2.2 Countermeasures recommended at Slide No.1 and No.2 and Its Requirements

Against the conditions which cause damage to NH10, the countermeasures shown in Table 2-1 are recommended. The requirements of the countermeasures are explained in the drawings attached to Appendix.

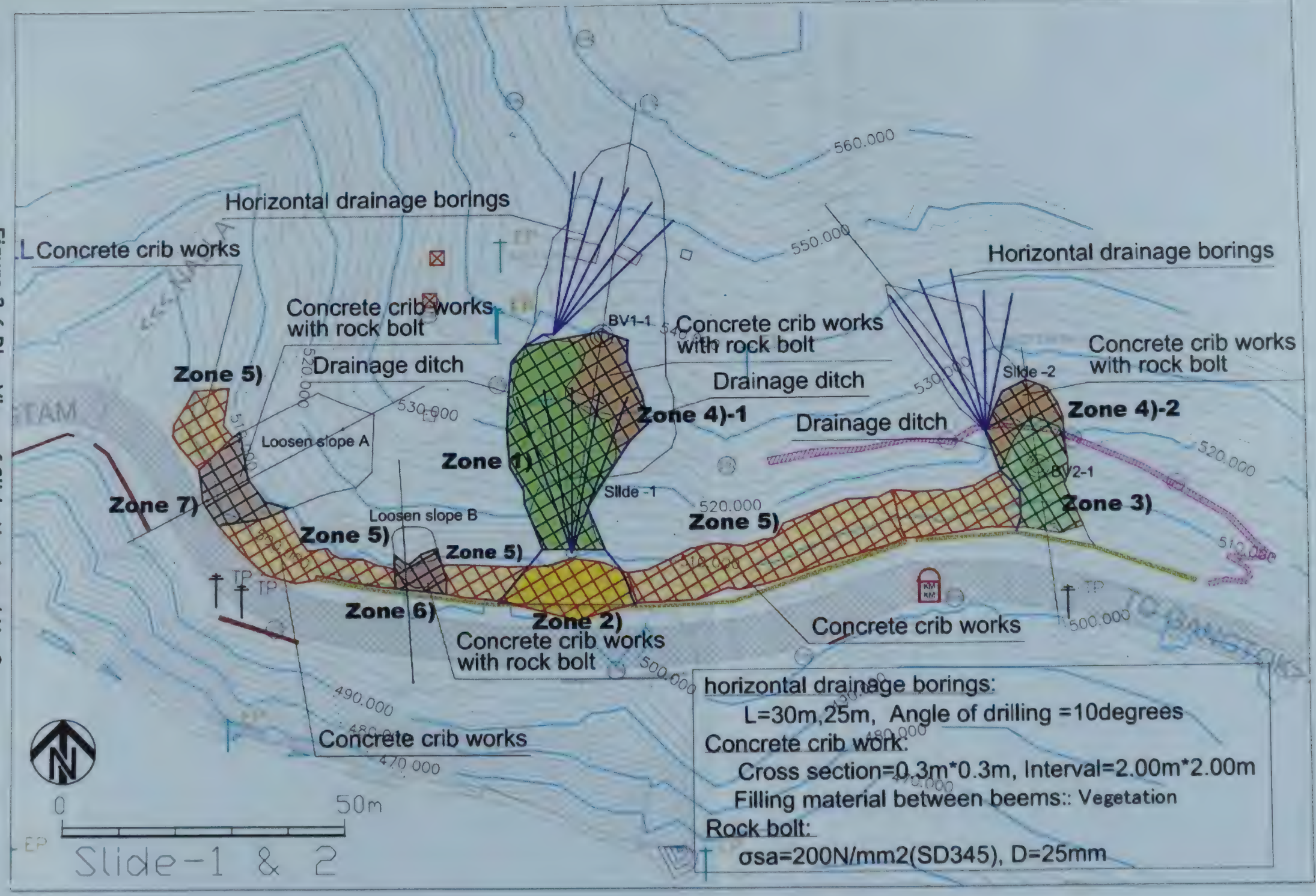
Table 2-1 Zones and Countermeasures

Zones	Purpose	Countermeasures
1) Upper slide area of Slide No.1	✓ To prevent further circular failure triggered by seepage pressure ✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works (b) Rock bolts (c) Horizontal drainage boring with drainage ditch to the toe
2) Lower slide area of Slide No.1	✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works
3) Slide area of Slide No.2	✓ To prevent further circular failure ✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works (b) Rock bolts (c) Horizontal drainage boring with drainage ditch to the toe
4) Gentle slopes where possible slides hide 4)-1: on Slide No.1 4)-2: on Slide No.2	✓ To prevent possible failure ✓ To retain the slope ✓ To prevent erosion ✓ To ease seepage pressure	(a) Cast-in-situ concrete crib works (b) Rock bolts (c) Horizontal drainage boring with drainage ditch to the toe
5) Road side steep slopes	✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works (b) Rounding of shoulder
6) Road side steep slope with possible deformation block	✓ To prevent possible surface failure ✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works (b) Rounding of shoulder (c) Rock bolts
7) Road side steep slope with large boulders and fallen rocks	✓ To remove large boulders and fallen rocks ✓ To prevent possible surface failure ✓ To retain the slope ✓ To prevent erosion	(a) Cast-in-situ concrete crib works (b) Rounding of shoulder (c) Rock bolts (d) Removal of large boulders and fallen rocks

The plan view of Slide No.1 and No.2 is shown in Figure 2-6 and the representative cross sections are from Figure 2-7 to Figure 2-11.

Figure 2-6 Plan View of Slide No.1 and No.2

A02-10



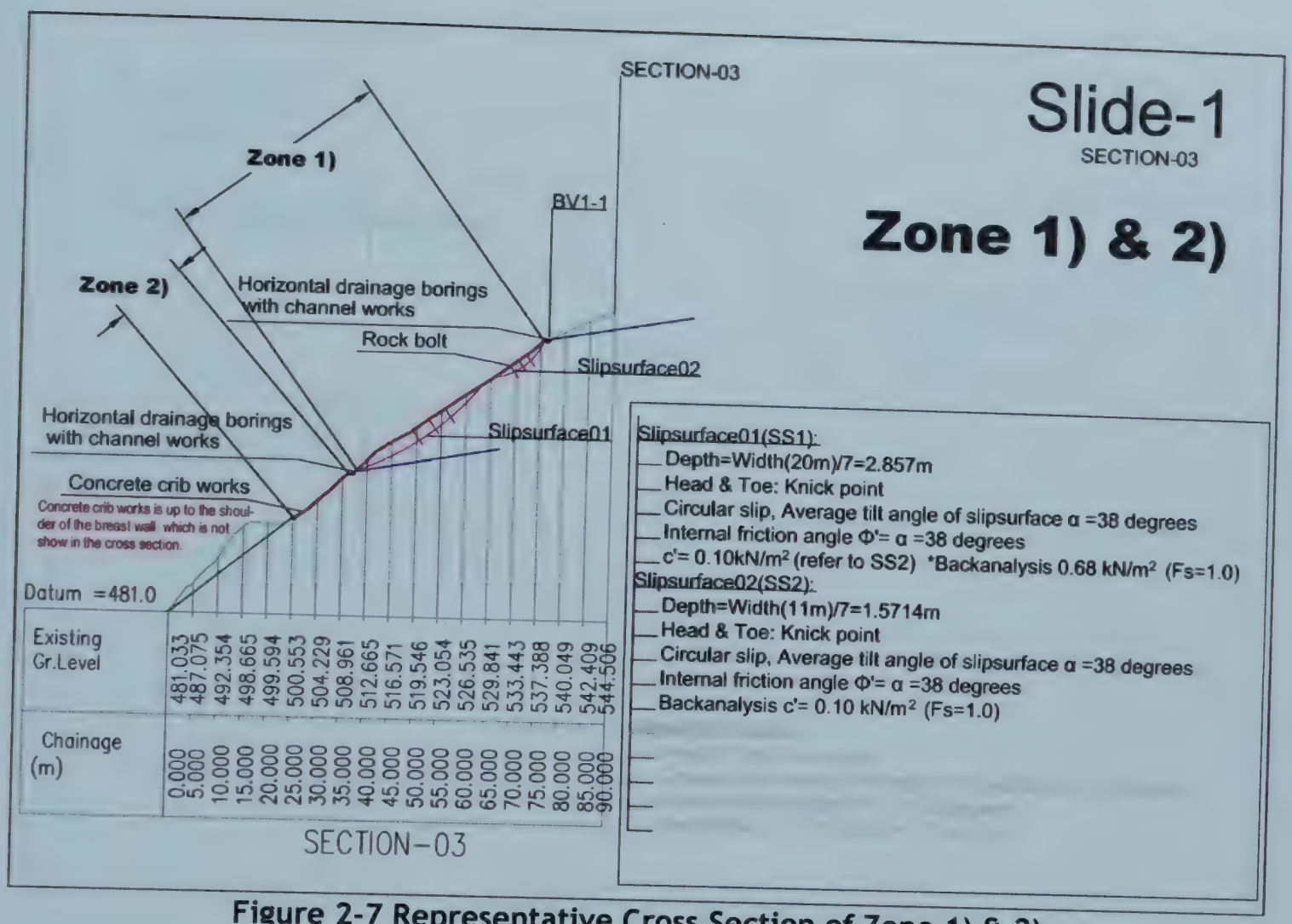


Figure 2-7 Representative Cross Section of Zone 1) & 2)

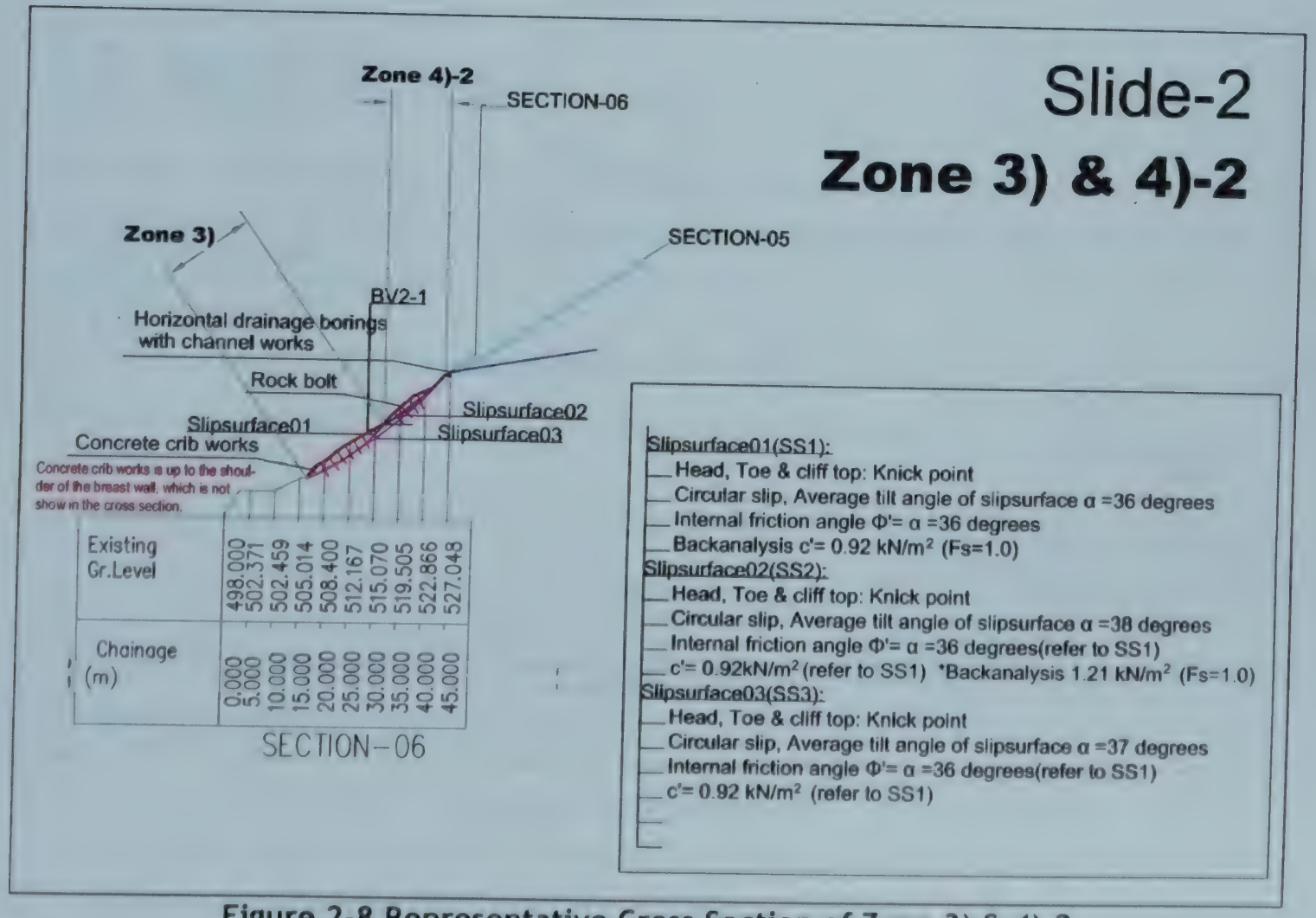


Figure 2-8 Representative Cross Section of Zone 3) & 4)-2

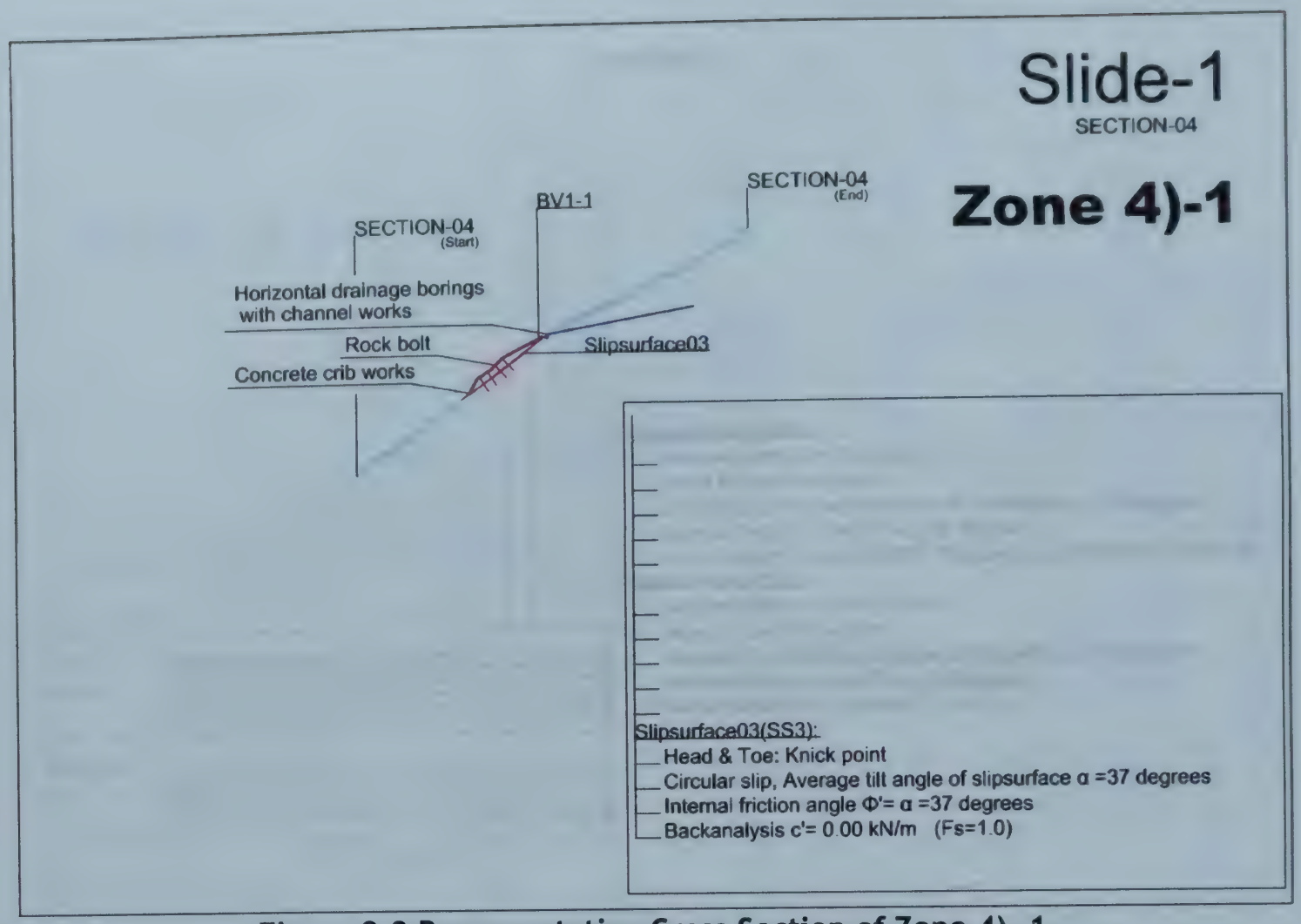


Figure 2-9 Representative Cross Section of Zone 4) -1

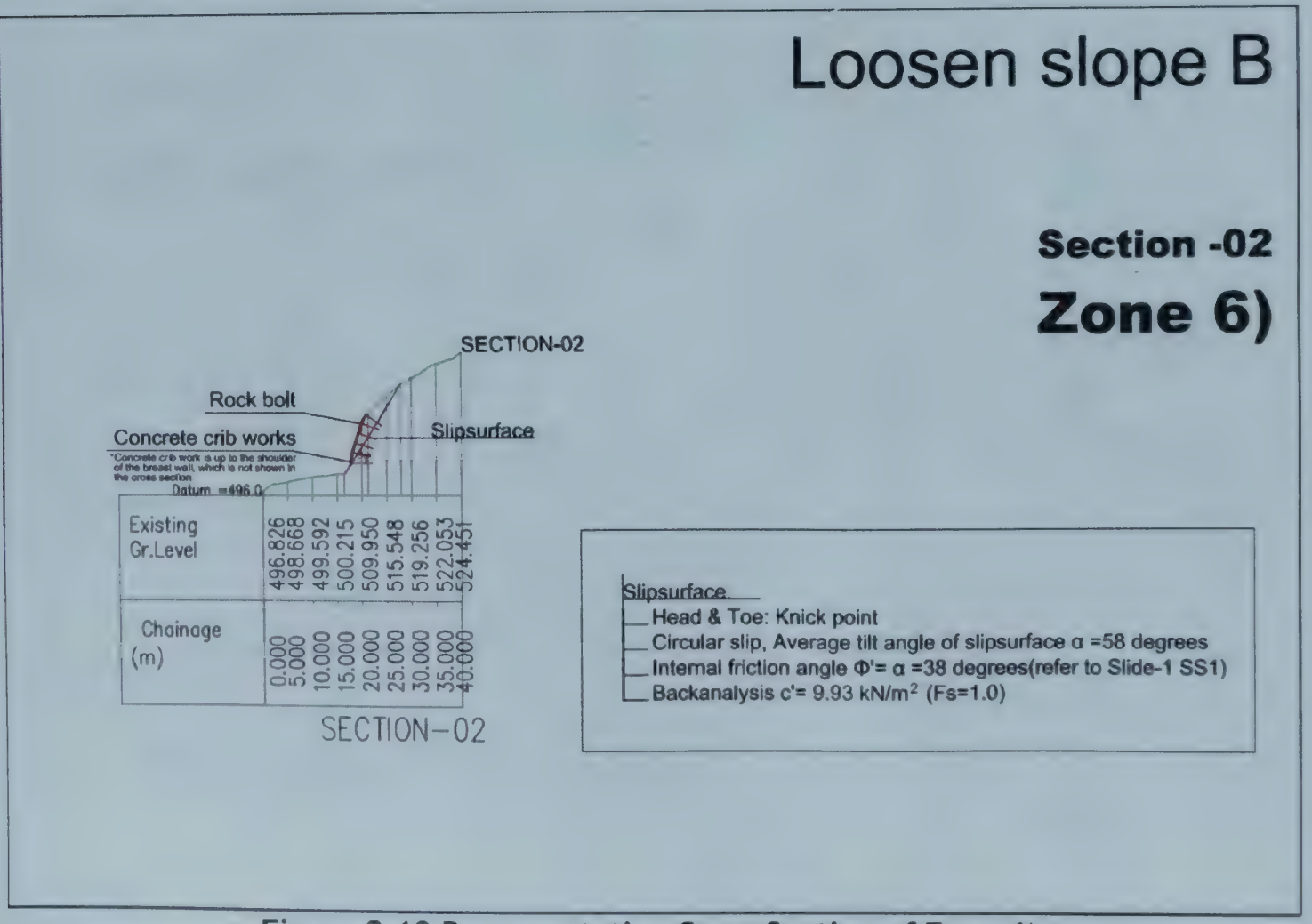


Figure 2-10 Representative Cross Section of Zone 6)

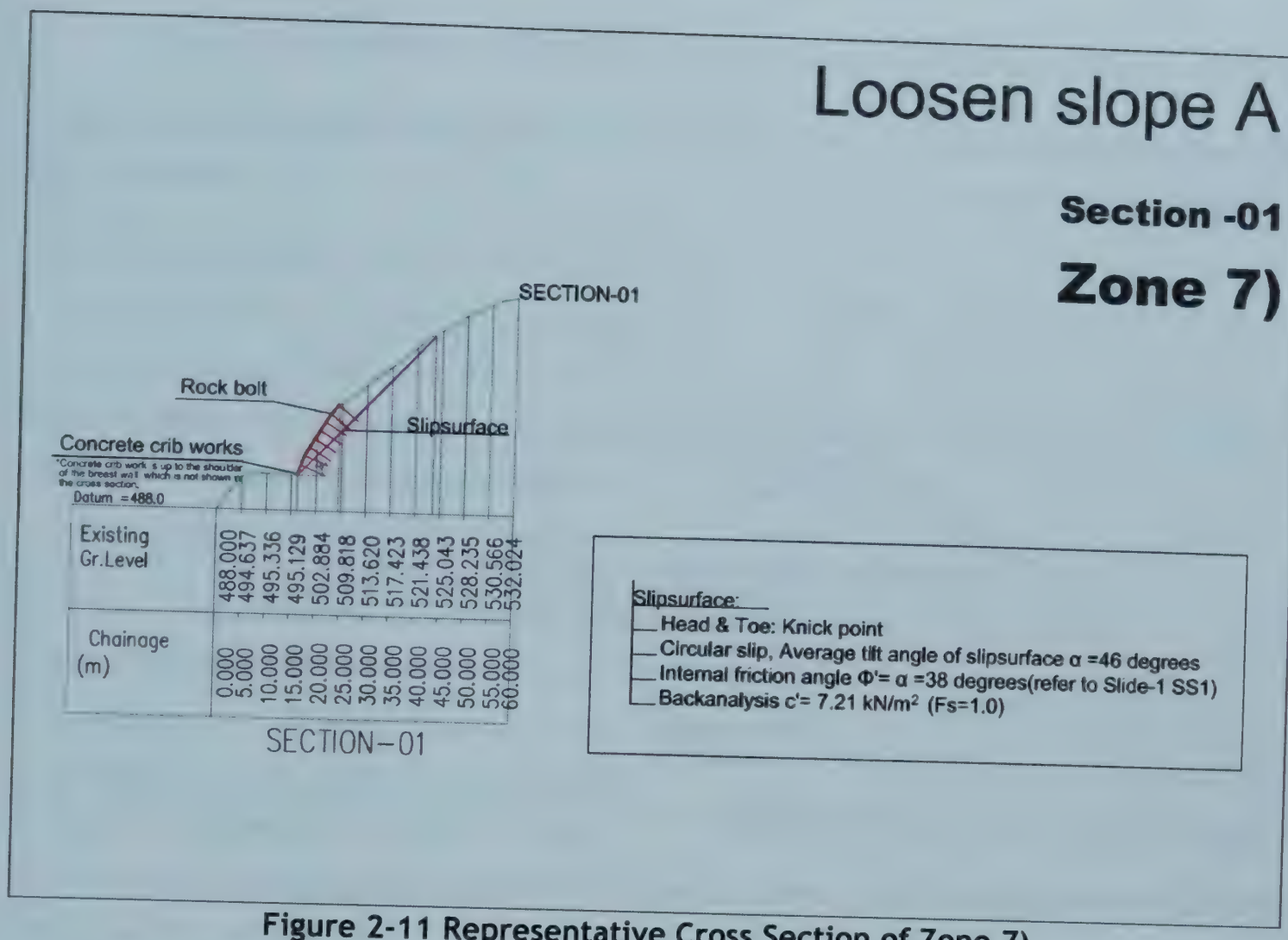


Figure 2-11 Representative Cross Section of Zone 7)

2.3 Outline of Design of Countermeasures

The details of design procedure are described in Chapter 4 in this report as well as the Draft Guidelines. For the countermeasures already introduced in India, the existing IRC guidelines should be referred.

For the zone 1), 2), and 3), unstable boulders should be removed and the surface of slope should be trimmed before installing the countermeasures.

2.3.1 Countermeasures for Zone 1)

At the zone 1), further slope failure may occur triggered by seepage water pressure. Thus the following countermeasures are required.

a) Horizontal drainage boring to ease seepage pressure
b) Cast-in-situ concrete crib works to retain the slope and to prevent erosion
c) Rock bolts to reinforce the slope.

Rock bolts should be installed at the intersections of frames of concrete crib works.

For the design of rock bolts, a set of cohesion and internal friction angle was set through the following consideration:

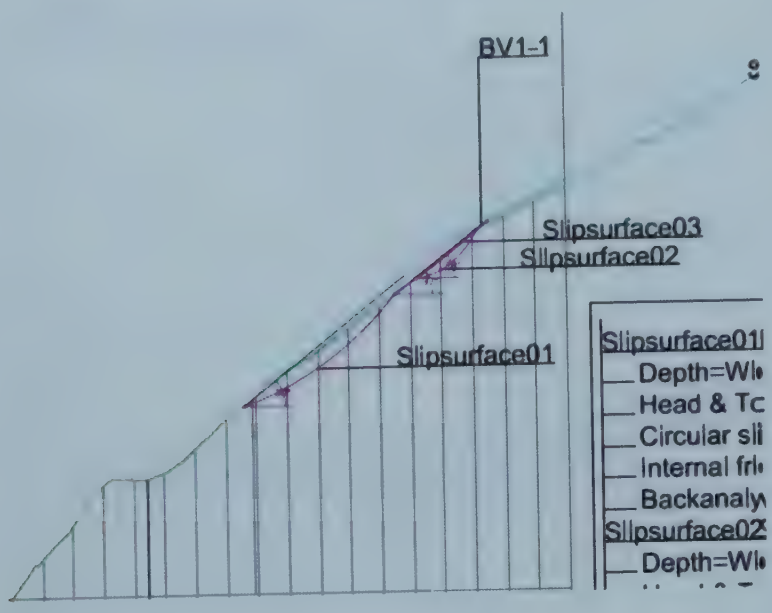


Figure 2-12 Estimation of Parameters

According to the geological conditions of the site, where massive boulder and cobbles are abundant, and the depths of the existing slides, a possible slip surface should run through shallower portions of the slope less than 3 meters deep from the ground.

From the cross section on the main survey line of Slide No.1, a couple of possible slip surfaces were assumed as shown in Figure 2-12, based on which average gradient of slip surface was examined and estimated as 38 degrees.

Then the internal friction angle of possible slip surface was set as 38 degrees. Through back analysis with the factor of safety as Fs=1.0, cohesion was estimated as 0.10 kN/m^2.

The slope stability analysis for the back analysis was performed under effective stress condition but without water and the same conditions are applied to the Slide No.2 and No.3 as well. For such steep slopes, assuming a factor of safety as Fs = 1.0 for analysis conditions with groundwater provides unrealistic strong strength of slip surface.

Note:
Groundwater condition for back analysis should be well-examined based on the conditions of the site and landslide. The factor of safety of Fs = 1.0 means the slope is in a limit equilibrium status, where the slope is on the boundary of being sliding and still.

The design method of concrete crib work is explained in the Draft Guidelines and additional information is given in Chapter 4.

2.3.2 Countermeasures for Zone 2)

At the zone 2), cast-in-situ concrete crib works are required to retain the slope and to prevent erosion.

2.3.3 Countermeasures for Zone 3)

At the zone 3), the similar type of slope failure with Zone 1) is expected. Thus the same countermeasures are required as follows:

a) Horizontal drainage boring to ease seepage pressure
b) Cast-in-situ concrete crib works to retain the slope and to prevent erosion
c) Rock bolts to reinforce the slope.

Rock bolts should be installed at the intersections of frames of concrete crib works.

For the design of rock bolts, a set of cohesion and internal friction angle was set through the following consideration:

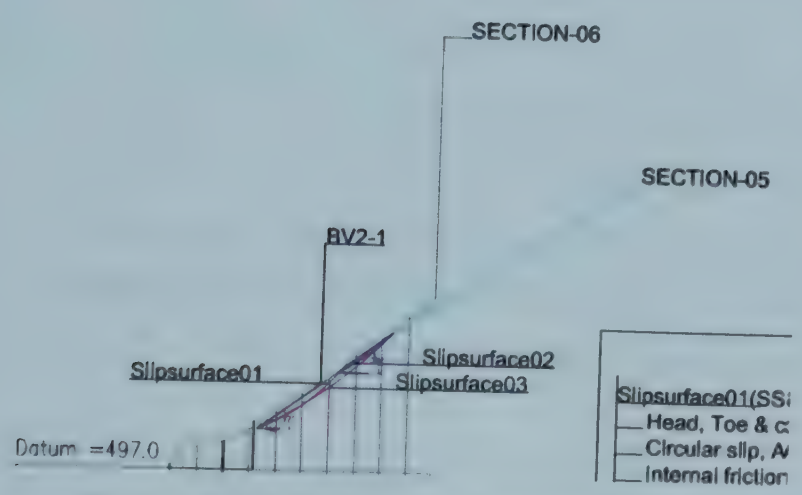


Figure 2-13 Estimation of Parameters

According to the geological conditions of the site, where massive boulder and cobbles are abundant, and the depths of the existing slides, a possible slip surface should run through shallower portions of the slope less than 3 meters deep from the ground.

From the cross section on the main survey line of Slide No.2, a couple of possible slip surfaces were assumed as shown in Figure 2-13, based on which average gradient of slip surface was examined and estimated as 36 degrees.

Then the internal friction angle of possible slip surface was set as 36 degrees. Through back analysis with the factor of safety as Fs=1.0, cohesion was estimated as 0.92 kN/m^2.

2.3.4 Countermeasures for Zone 4)

The zone 4) is composed of 2 separate zones of 4)-1 and 4)-2. The zone 4)-1 is adjacent

to the zone 1) and the zone 4)-2 is on the upper slope of the zone 3). At the zone 4)-1, potential slope failure similar to the zone 1) may occur. On the other hand, a potential slope failure of the zone 3) may extend to the zone 4)-2. Thus the following countermeasures are required for both zone 4)-1 and 4)-2.

a) Cast-in-situ concrete crib works to retain the slope and to prevent erosion
b) Rock bolts to reinforce the slope
c) Horizontal drainage boring to ease seepage pressure (to utilize the same works for the zone 1) and zone 3) as much as possible)

Rock bolts should be installed at the intersections of frames of concrete crib works.

For the design of rock bolts, a set of cohesion and internal friction angle was selected for the zone 4)-1 and 4)-2 as follows:

For Zone 4)-1

- ✓ Internal friction angle: the average gradient of possible slip surface of 37 degrees was adopted, as shown in Figure 2-9.
- ✓ Cohesion: through back analysis of slope stability analysis with a factor of safety of Fs=1.0.

For Zone 4)-2

- ✓ Since the same slip surface runs through from the zone 3) to zone 4)-2, the same set of internal friction angle and cohesion with the zone 3) was adopted.

2.3.5 Countermeasures for Zone 5)

At the zone 5), cast-in-situ concrete crib works are required to retain the slope and to prevent erosion. At the same time, the shoulder of the slope should be rounded to ease loosening.

2.3.6 Countermeasures for Zone 6)

At the zone 6), potential surface failure may occur. Thus the following countermeasures are required along with rounding of the shoulder.

a) Cast-in-situ concrete crib works to retain the slope and to prevent erosion
b) Rock bolts to reinforce the slope

Rock bolts should be installed at the intersections of frames of concrete crib works.

For the design of rock bolts, a set of cohesion and internal friction angle was set as follows:

- ✓ Internal friction angle: The gradient of possible slip surface was too steep to be adopted as an internal friction angle, as shown in Figure 2-10. Due to similarity of geological conditions, internal friction angle of Zone 1) as 38 degrees was adopted.
- ✓ Cohesion: through back analysis of slope stability analysis with a factor of safety of Fs=1.0.

2.3.7 Countermeasures for Zone 7)

At the zone 7), in addition to the countermeasures for the zone 6), removal of unstable boulders and fallen rocks are required.

For the design of rock bolts, a set of cohesion and internal friction angle was set as follows:

- ✓ Internal friction angle: The gradient of possible slip surface was too steep to be adopted as an internal friction angle, as shown in Figure 2-11. Due to similarity of geological conditions, internal friction angle of Zone 1) as 38 degrees was adopted.
- ✓ Cohesion: through back analysis of slope stability analysis with a factor of safety of Fs=1.0.

3 Slide No.3 at Km 71 + 700

3.1 Conditions of Slope and Landslides

The conditions of the site are summarized in the following report:

(2) Advisory Report for Landslide Sites of NH10 in the State of Sikkim for Selecting Site for Pilot Civil Work (31st January - 2nd February) dated on the 25th of February, 2019.

The landslide at Slide No.3 has three significant features:

1) An upper escarpment with a height of 150 meters and almost vertical gradient,
2) A lower steep slope with a height of 100 meters, and
3) A gravity deformation block located on the west of the steep slope of 2).

The 1) upper escarpment provides fallen rocks to the lower steep slope and eventually NH10 from the loosened outcrop of the bedrock of phyllite with fragile bands of sericite schist and quartzite.

The 2) lower steep slope is composed of 4 parts; 2)-1 wedge failure and unstable block of gravity deformation at the shoulder, 2)-2 steep slope with loosened bedrock in the upper middle, 2)-3 moderately steep slope in the lower middle, and 2)-4 steep slope of loosened bedrock at the toe.

The gradient of the slope is generally steep and varies between 30 to 50 degrees. Throughout the slope, a group of slope failures involving debris and loosened shallower parts of bedrock governs. The surface of the slope is mainly loosened bed rock but thin debris covers some parts.

The 3) gravity deformation block does not cause clear damage to NH10. But minor deformation of the breast wall and slight uplift of the pavement was observed. The slope failures at the 2) lower steep slope corresponds to the right side flank of the block. Thus the activity of this gravity deformation block might facilitate the failures at the adjacent slope.

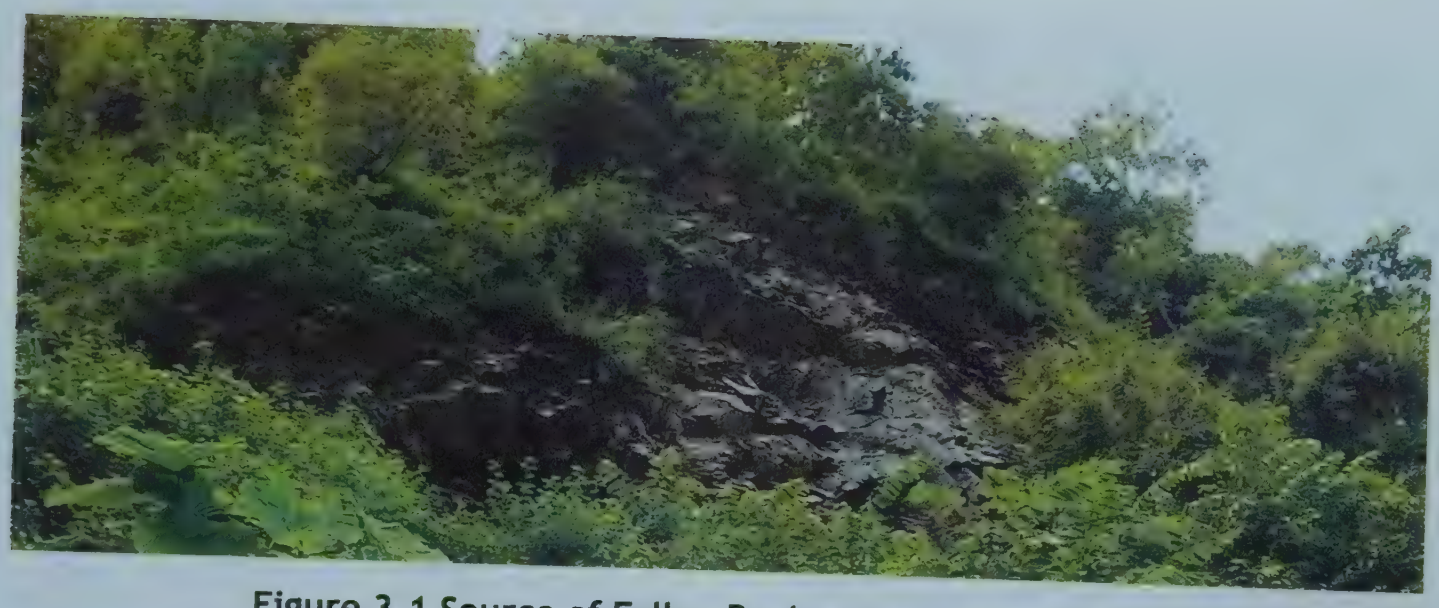


Figure 3-1 Source of Fallen Rocks at 1) Upper Escarpment

Figure 3-2 View of 2) Lower Steep Slope (Left) and Loosened Bedrock (Upper Right)

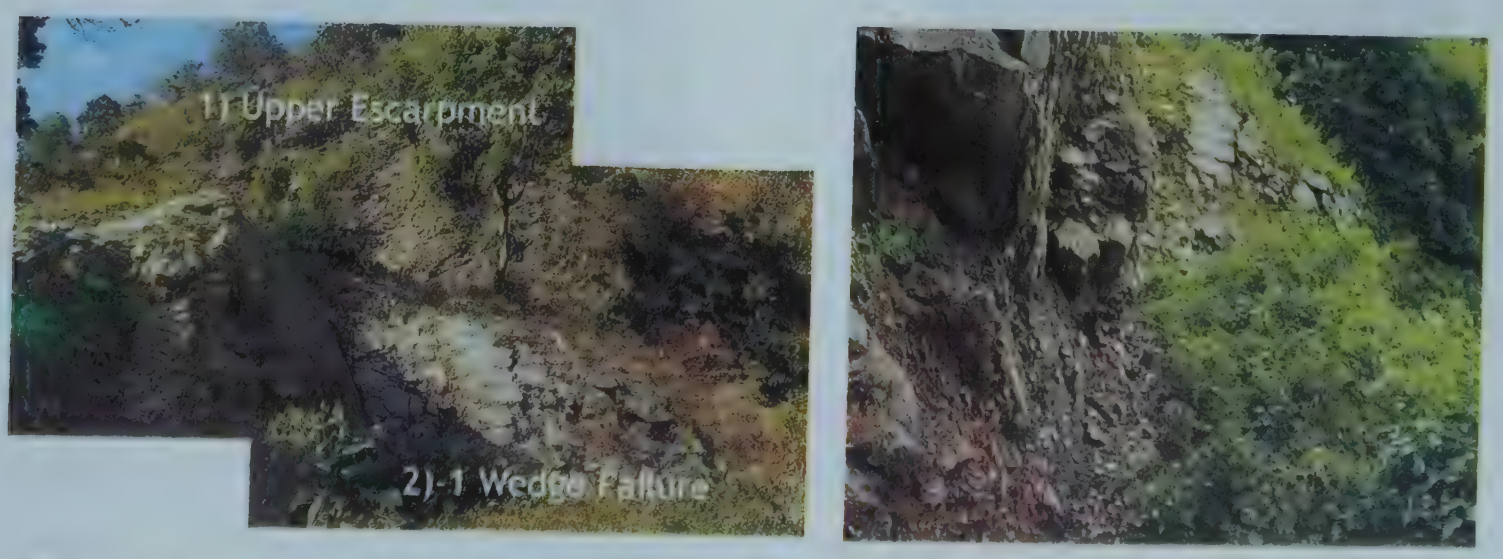


Figure 3-3 Upper Escarpment and Wedge Failure in January 2019 (Left) and Unstable Condition of Wedge Failure in June 2019 (Right)

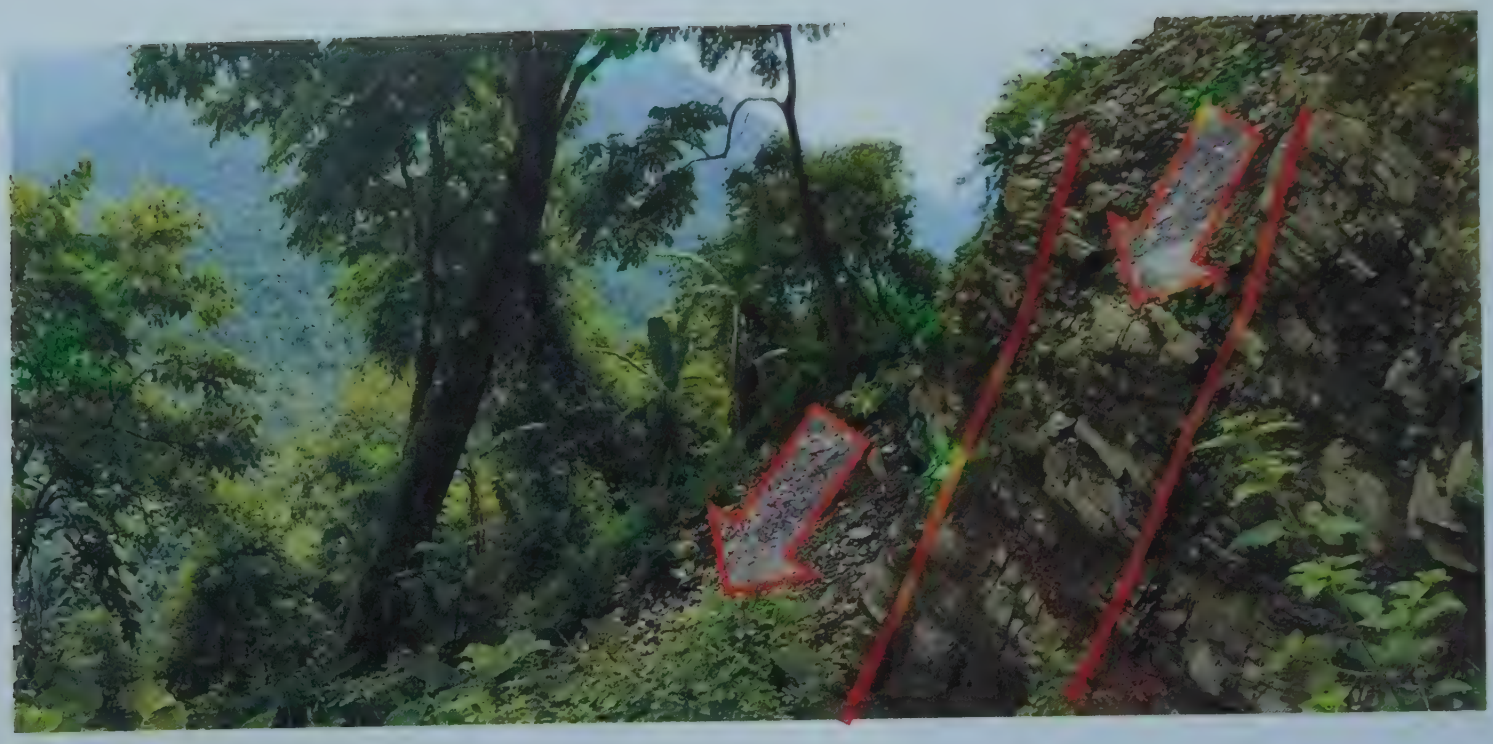


Figure 3-4 Loosened Bedrock at Wedge Failure, Suggesting Downward Deformation

Figure 3-5 Toe of Gravity Deformation of 2)-1 (Left) and Loosened Bedrock Covered by Thin Debris at the Toe of Slope (Right)

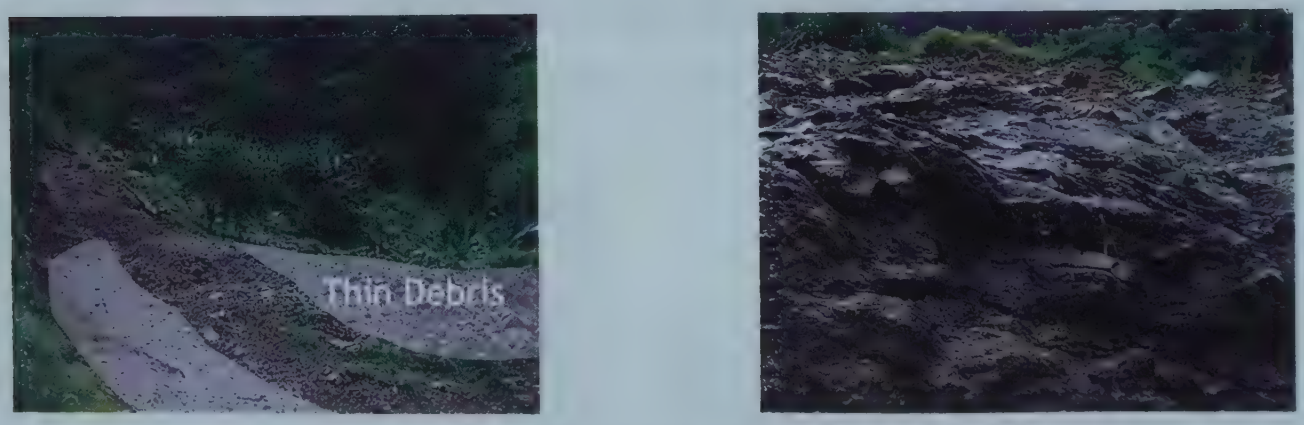


Figure 3-6 Middle Slope and Loosened Bedrock at Toe with Groundwater Spring (Right)

To clarify the conditions of Slide No.3, drilling investigation and cross section survey were requested in the letter issued by JICA Expert with a reference No. JXPTHE-008/2019, dated on the 15th of April, 2019.

3.2 Countermeasures recommended at Slide No.3 and Its Requirement

Against the conditions which cause damage to NH10, the countermeasures shown in Table 3-1 are recommended. The requirements of the countermeasures are explained in the drawings attached to Appendix.

Table 3-1 Conditions and Countermeasures

Conditions	Purpose	Countermeasures
1) the upper escarpment which provide fallen rocks	✓ To prevent rock fall	(a) Rock fall protection net with rock bolts
2)-1 wedge failure and unstable block of gravity deformation at the shoulder	✓ To prevent gravity deformation and related slope failures. ✓ To prevent deep-seated landslide which may develop from the gravity deformation.	(a) Ground anchors (b) Retaining wall (c) Horizontal drainage boring with drainage ditch to the toe
2)-2 steep slope with loosened bedrock in the upper middle	✓ To prevent slope failures involving loosened rock and debris ✓ To stable loosened rock slope ✓ To prevent erosion of slope	(a) Removal of debris (b) Shotcrete (c) Rock bolts
2)-3 moderately steep slope in the lower middle	✓ To prevent slope failures involving rock surface and debris ✓ To prevent erosion of slope	(a) Removal of debris (b) Shotcrete
2)-4 steep slope of loosened bedrock at the toe	✓ To prevent slope failures involving loosened rock and debris ✓ To stable loosened rock slope ✓ To prevent erosion of slope ✓ To reduce groundwater pressure	(a) Removal of debris (b) Cast-in-situ concrete crib works (c) Rock bolts (d) Horizontal drainage boring with drainage ditch to the toe

The pilot project is subject to the limitation that the works adopted in the pilot project should be implemented by Indian contractors with technical supports by JICA Experts. Due to this limitation, the most favorable countermeasure to the slope was not adopted. Shotcrete crib works with embedded formwork will provide wide application to high and steep slope like the Slide No.3. But the embedded formwork should be imported from Japan and thus shotcrete crib works were not adopted. Instead, cast-in-situ concrete crib works, which require independent formwork to install before and to remove after casting concrete, was adopted for 2)-4 steep slope at the toe, and applying shotcrete was adopted for 2)-2 upper middle and 2)-3 lower middle of the lower steep slope.

Shotcrete crib works with embedded formwork are versatile slope protection and highly applicable for the roadside slopes in India. It can even follow irregular shapes of natural slopes. Thus introducing shotcrete crib works in the pilot project may be an option worth to consider. To introduce shotcrete crib works, however, we need to overcome at least two challenges; procurement and import by NHIDCL and material supply by Japanese

supplier. If Japanese suppliers offered bids to the tender by NHIDCL to procure the embedded formwork and if NHIDCL procured and imported the embedded formwork from Japan, shotcrete crib works would able to be adopted. But these assumption inevitably include uncertainties in the progress of the pilot project. Due to the limitation of the schedule of the pilot project, we have no spare time to allow such uncertainties and thus we recommend the countermeasures as shown in Table 3-1.

The plan view of Slide No.3 is shown in Figure 3-7 and representative cross section is in Figure 3-8. The plan view and cross sections used for the recommendation are partially rectified and adjusted with the satellite data ALOS World 3D[1] provided from the Japanese space agency JAXA[2] and field verification on site with laser range finders.

For the 3) gravity deformation block located on the west of the 2) lower steep slope, monitoring of deformation is required. Thus instruction to the Contractor should include installation and observation of monitoring equipment for surface and underground movement.

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1. https://www.eorc.jaxa.jp/ALOS/aw3d30/index_j.htm

2 Japan Aerospace Exploration Agency https://global.jaxa.jp/

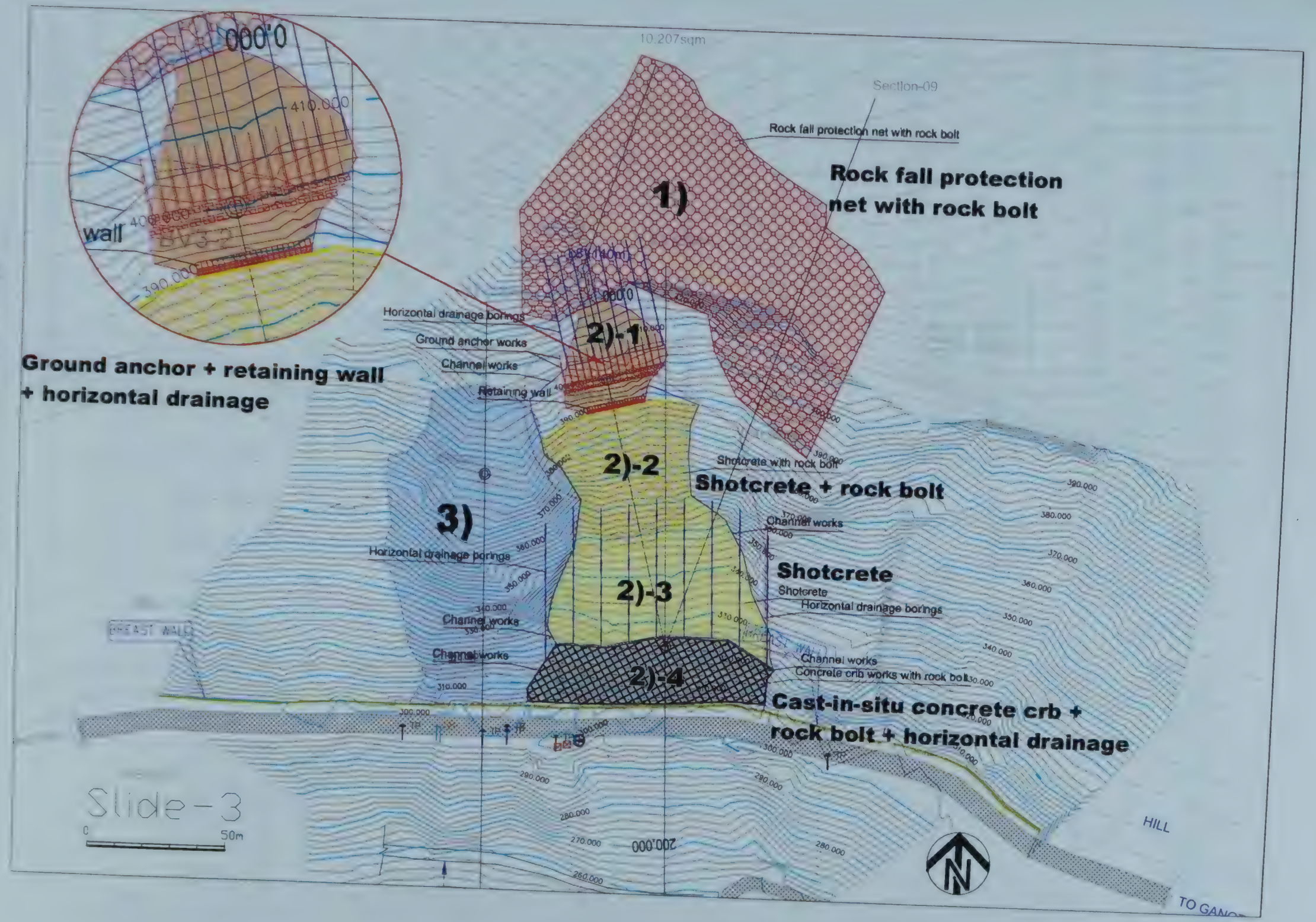

Figure 3-7 Plan View for Slide No.3

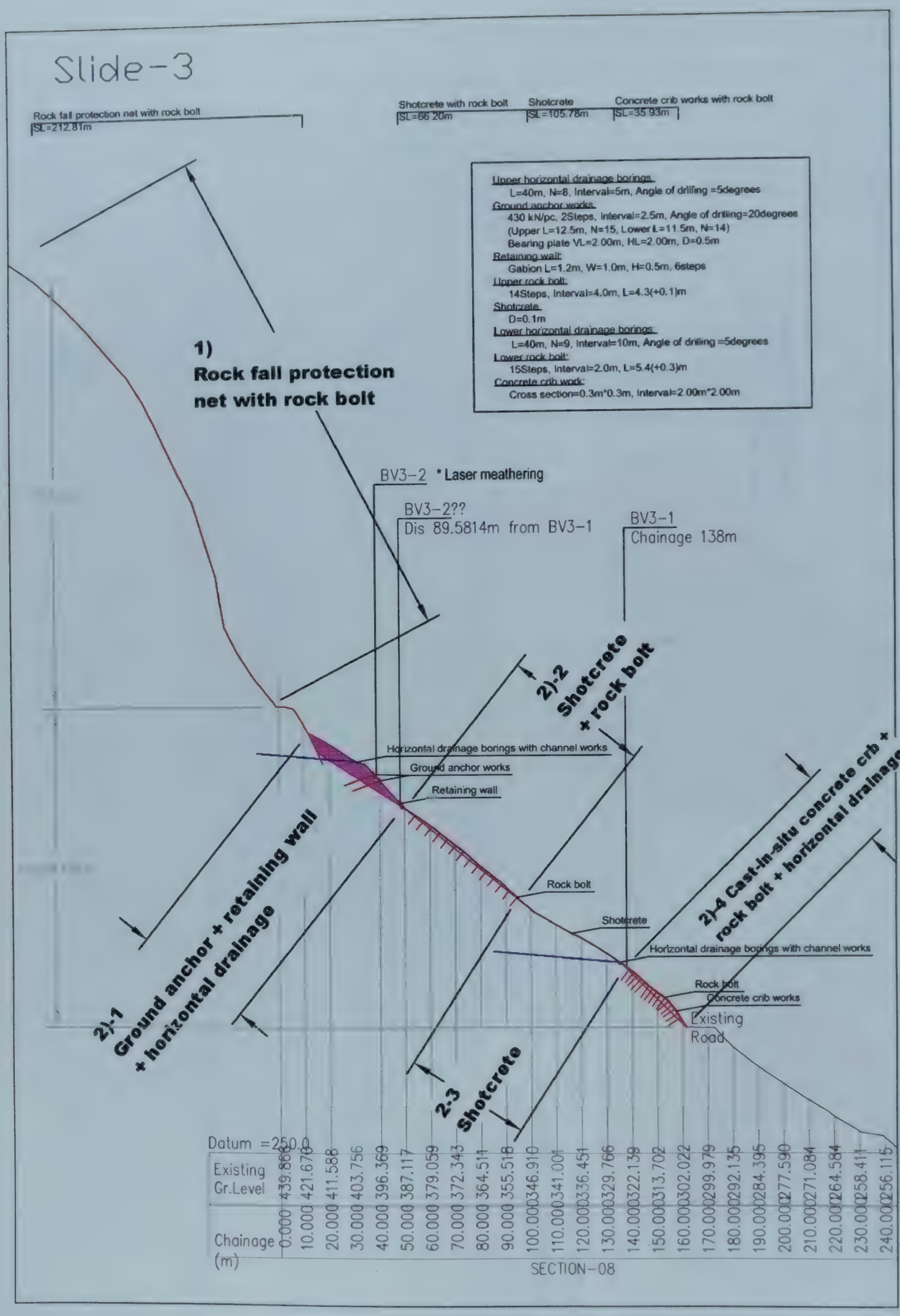

Figure 3-8 Representative Cross Section for Slide No.3

3.3 Outline of Design of Countermeasures

The details of design procedure are described in Chapter 4 in this report as well as the Draft Guidelines. For the countermeasures already introduced in India, the existing IRC guidelines should be referred

For all sections of the lower steep slope, debris should be removed before installing the countermeasures.

3.3.1 Countermeasures for 1) Upper Escarpment

Rock fall protection net with rock bolts is available in Indian market with existing guideline of HRB-SR23[3] to be referred in DPR preparation. Thus in this recommendation, method of detailed design is not provided.

The estimated slant height is more than 210 meters and vertical height is more than 150 meters. However, the survey results available as of the recommendation lack necessary information for quantity calculation of rock fall protection net.

The situation of the upper escarpment is as shown in Figure 3-1. The surface of the escarpment is densely cracked and loosened.

According to the fallen rocks observed at the shoulder of the escarpment, the expected size of fallen rocks from the escarpment is Φ 300 ~ 400 mm. The rock fall protection net to be applied should trap this size of fallen rocks or smaller. The intervals of the cracks on the escarpment suggest a maximum size of fallen rocks as Φ 1,000 mm, which should be retained at the original positions. The objective of the rock fall protection net is to retain detached fallen rocks at the source or, if it is smaller, lead them to the toe of the escarpment without letting them escape from the restriction of the net.

On the other hand, the loosened condition of the surface requires additional reinforcement. At the same time, the rock fall protection net to be applied should be anchored on to the escarpment, to support its weight and to retain its position on the surface of the escarpment. For the said two objectives, rock bolting for certain intervals should be applied.

In addition to rock fall protection nets, along the toe of the escarpment, it is

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[3] State of the Art: Design and Construction of Rockfall Mitigation Systems: IRC 2014

recommended to install drainage ditches with net fence along with it. Net fence is supposed to protect patrol person and trap and prevent small fallen rocks from falling to NH10 in case it escapes through the rock fall protection net.

3.3.2 Countermeasures for 2)-1 Shoulder of Lower Steep Slope

At the shoulder of the lower steep slope, the unstable block of gravity deformation lies and requires countermeasure to increase stability.

3.3.2.1 Setting Mechanical Parameters

For the design of countermeasures, the mechanical parameters, a set of cohesion and internal friction angle, of the loosened zone of the block of gravity deformation are required. The set of cohesion and internal friction angle of the loosened zone were estimated with the following procedure.

(1) Setting Internal Friction Angle from Gradient of Slope

A slope at a trace of a slope failure or a landslide often corresponds to a slip surface of the slope failure or landslide. The average gradient of slope of the yellow shaded section in Figure 3-9 is 40 degrees, which corresponds to a gradient of slip surface of the slope failures which shaped the slope. A gradient of a slip surface often corresponds to an internal friction angle of the slip surface. Thus the internal friction angle of 40 degrees was set for the dominant slope failure of the lower steep slope.

(2) Estimation of Area of Gravity Deformation

Then the area of the block of gravity deformation was estimated.

The dominant slope failure may affect the shoulder of the slope and develop a head scarp there. The gradient of the head scarp is similar to the angle of active failure, which is calculated from the formula below:

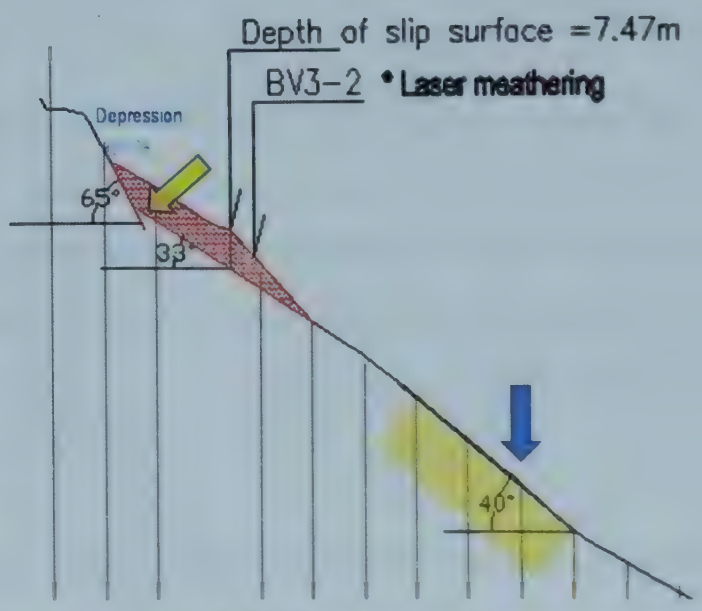

Figure 3-9 Estimation of Parameters

$A = 45 + \varphi/2$ degrees

where A: Angle of active failure (degree)
φ: internal friction angle (degree)

Thus a line of head scarp was extended from the shoulder of the block.

Then a width of depression at the head of a landslide roughly corresponds to a depth of slip surface[4]. Thus a point was placed just below the lower scarp of the depression at the depth of the width of it, which is pointed by the yellow arrow in Figure 3-9. Then a line connecting the point and the toe of the block was drawn and extended to the line of head scarp. This line corresponds to the loosened zone of the gravity deformation block, the gradient of which is 33 degrees.

(3) Estimation of Cohesion and Internal Friction Angle of Loosened Zone

The internal friction angle of the loosened zone was set to 33 degrees from the gradient of it. The cohesion of the loosened zone was decided through the back analysis of the slope stability analysis of the gravity deformation block, in which the factor of safety as Fs=1.0 was set for the calculation.

3.3.2.2 Ground Anchor Works

Ground anchor works was designed with the set of cohesion and internal friction angle estimated in the previous section, with a design factor of safety of Fs=1.2.

3.3.2.3 Retaining Wall

To provide stability to the area under the lower array of ground anchor, gabion retaining wall is planned at the toe of the block of gravity deformation.

3.3.2.4 Horizontal Drainage Boring

The slope stability analyses performed in Chapter 3.3.2.1 and 3.3.2.2 were calculated under effective stress condition without setting groundwater table because no information regarding groundwater was available and analysis with groundwater table may provide excessive design.

The design of the ground anchor does not allow groundwater infiltration to the block. Thus groundwater drainage works are required. Horizontal drainage borings should be

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[4] D.M.Cruden (1991) Observation of Graben geometry in landslides

aligned above the upper array of ground anchor, in the aim of dewatering the ground behind. A surface drainage system should be arranged to make the drained water flow to the roadside ditches or other appropriate water system.

3.3.3 Countermeasures for 2)-2 Upper Middle Part of Lower Steep Slope

At the upper middle part of the lower steep slope, loosened bedrock appears at the surface. Thus the slope should be reinforced by rock bolts. In addition, in the aim of preventing erosion and peeling off of fragment of rock, applying shotcrete is required.

For the design of rock bolts, a set of cohesion and internal friction angle was set as follows:

- ✓ Internal friction angle: the gradient of the slope of 40 degrees was adopted
- ✓ Cohesion: through back analysis of slope stability analysis with a factor of safety of Fs=1.0.

3.3.4 Countermeasures for 2)-3 Lower Middle Part of Lower Steep Slope

At the lower middle part of the lower steep slope, the slope becomes gentler than the upper middle part and loosened part of rock was not confirmed. For this section, rock bolts are not required. For preventing erosion however, applying shotcrete is required

3.3.5 Countermeasures for 2)-4 Loosened Bedrock at Toe

At the toe, loosened bedrock of phyllite with bands of sericite schist and quartzite appears with groundwater springs at the boundary of overlying debris and the bedrock. Thus the following countermeasures are required.

a) Horizontal drainage boring to ease pressure of groundwater
b) Cast-in-situ concrete crib works to retain the slope
c) Rock bolts to reinforce the slope.

Rock bolts should be installed at the intersections of frames of concrete crib works.

The set of cohesion and internal friction angle for the design of rock bolt is the same as estimated in Chapter3.3.3.

The design method of concrete crib work is explained in the Draft Guidelines and additional information is given in Chapter 4.

4 Procedure of Design

This section describes the overall procedure to select and design the countermeasures for NH1O at Slide No.1, Slide No.2 and Slide No.3. Along with knowledge in Guidelines, Engineers of Project would like to provide more practical imagine by giving design procedures at specific sites.

Furthermore, the quality of design works for landslide countermeasures greatly depends on the quality of topographical survey, drilling and core samples as well as existing conditions of landslide areas.

However, at the time of submitting this Recommendation, the topographical survey and drilling survey have been carrying out and updating. Thus, the design in this Recommendation is based on the results of site visit, rectifying data, analysis and design works

General Design Procedure is expressed in Figure 4-1. Design procedures for countermeasures of ground anchor, rock bolt and quantity estimation are in Figure 4-2, Figure 4-3, and Figure 4-4, respectively. Figure 4-5 shows an example of quantity estimation and arrangement of frames for concrete crib works for reference.

DESIGN PROCEDURE FOR LANDSLIDE COUNTERMEASURES OF NH10

Site visit to check the existing conditions

Prior to site visit, map reading shall be performed to have overall imagine of slide area and critical locations to check at the site

- Deformation of existing structures at site such as road pavement, breast wall including crack, bulging, tilting, settlement, seepage water, opening joint and any unusual deformation;
- Conditions of outcrop (if available) such as looseness, weathering, erosion, seepage, vegetation cover, evidences of past sliding/movement;
- Conditions of landslide area Slide No. 1, Slide No. 2, Slide No. 3 and potential sliding mass nearby such as:
  - looseness, weathering, erosion, collapse of slopes;
  - matrix of debris or colluvium;
  - possibility of continuous failure by confirmation of evidence in slide area such as re-sliding of debris including boulders, instability and enlargement of boundary of landslide block, the elevation of soil pipe, earth pressure from the upper slope of failure;
  - estimation of failure surface depth
- Evaluation of possible countermeasures that are able to be applied at sites;
- Checking the drilling core samples (if available and reliable) to confirm depth of slip surface

Back Analysis to determine design parameter of slip surface

Based on the results of site visit and rectified topographical map, cross section for back analysis shall be prepared.

- Set up the Factor of Safety Fs = 1.00 to establish the relationship between internal friction angle ϕ' and cohesion c' on the slip surface of the minimum Factor of Safety
- Depending on the site evidences such as head and toe of slide, opening crack, collapsed area and soil matrix, fix value of internal friction angle ϕ', then derive cohesion c' from back analysis.

Confirmation the depth of slip surface

If the drilling core samples at sites are not sufficient or reliable enough to confirm the depth of slip surface, Engineers shall decide to find out the depth by evaluation of analysis results and site evidences

- From the collapsed area at site, specify the range of analyzed area in the model for analysis to find out the slip surface that needs the maximum prevention force (Max Pf) by calculation;
- Compare the depth of surface found out by calculation with the estimated depth by the site evidences;
- Determine the slip surface depth and type for design countermeasures

Countermeasure selection and design

For detailed design of countermeasures, cross sections for design along the protection area of landslide shall be prepared.

- Based on the site conditions, depth of failure surface and prevention force needed to stabilize the slope, appropriate countermeasures shall be selected and designed
  - Concrete crib works with rock bolt
  - Ground anchors
  - Rock fall net
  - Horizontal drainage borings

Design of Ground anchors		Design of concrete crib works with rock bolts		Design of rock fall net		Design of horizontal drainage borings	
Design for stability and construction	Quantity Estimation	Design for stability and construction	Quantity Estimation	Design for stability and construction	Quantity Estimation	Design for stability and construction	Quantity Estimation

Figure 4-1 General design procedure for landslide countermeasures of NH10

4.1 Design procedure of Ground Anchor

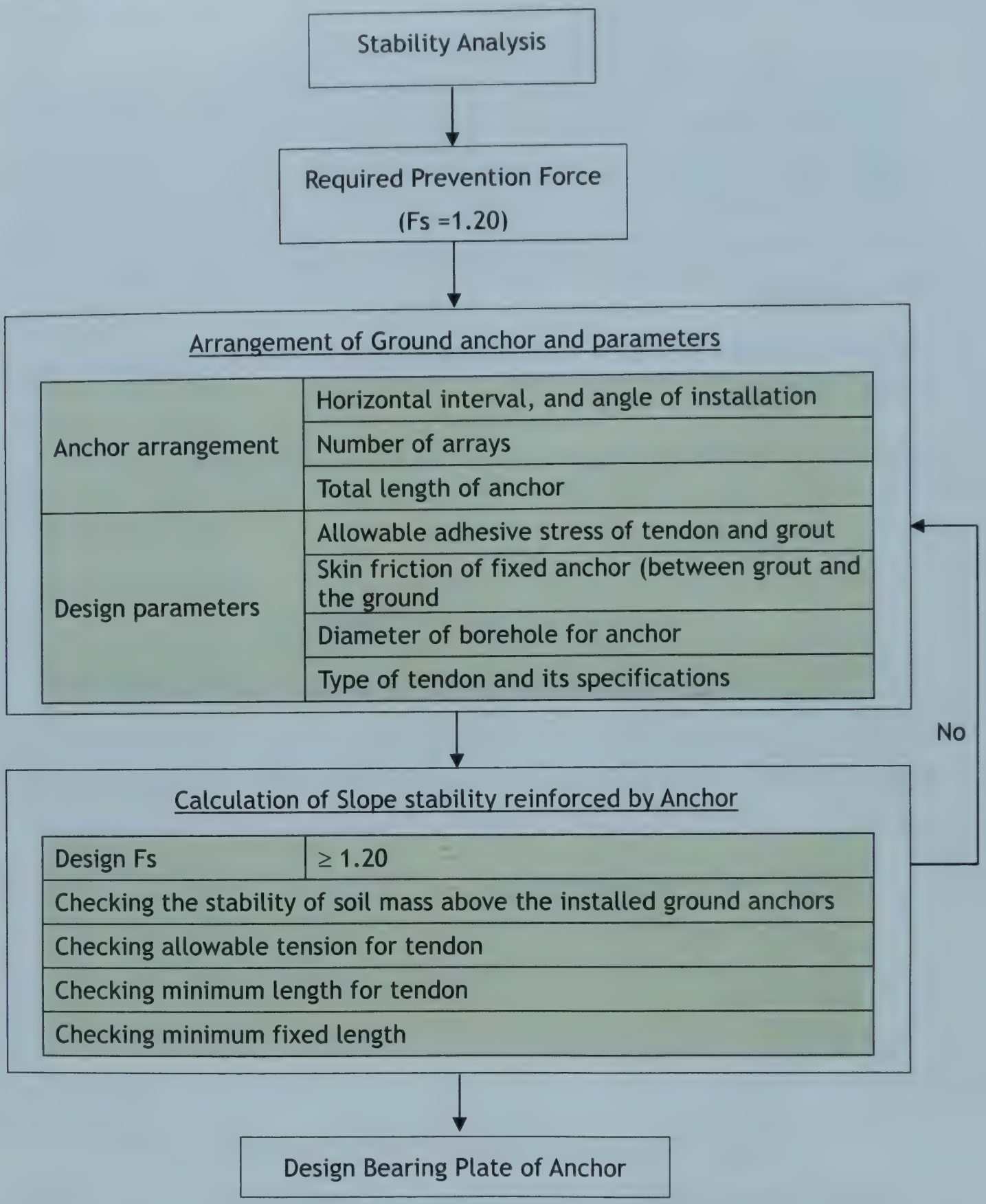


Figure 4-2 Design procedure of Ground Anchor

4.2 Design procedure of Rock Bolt

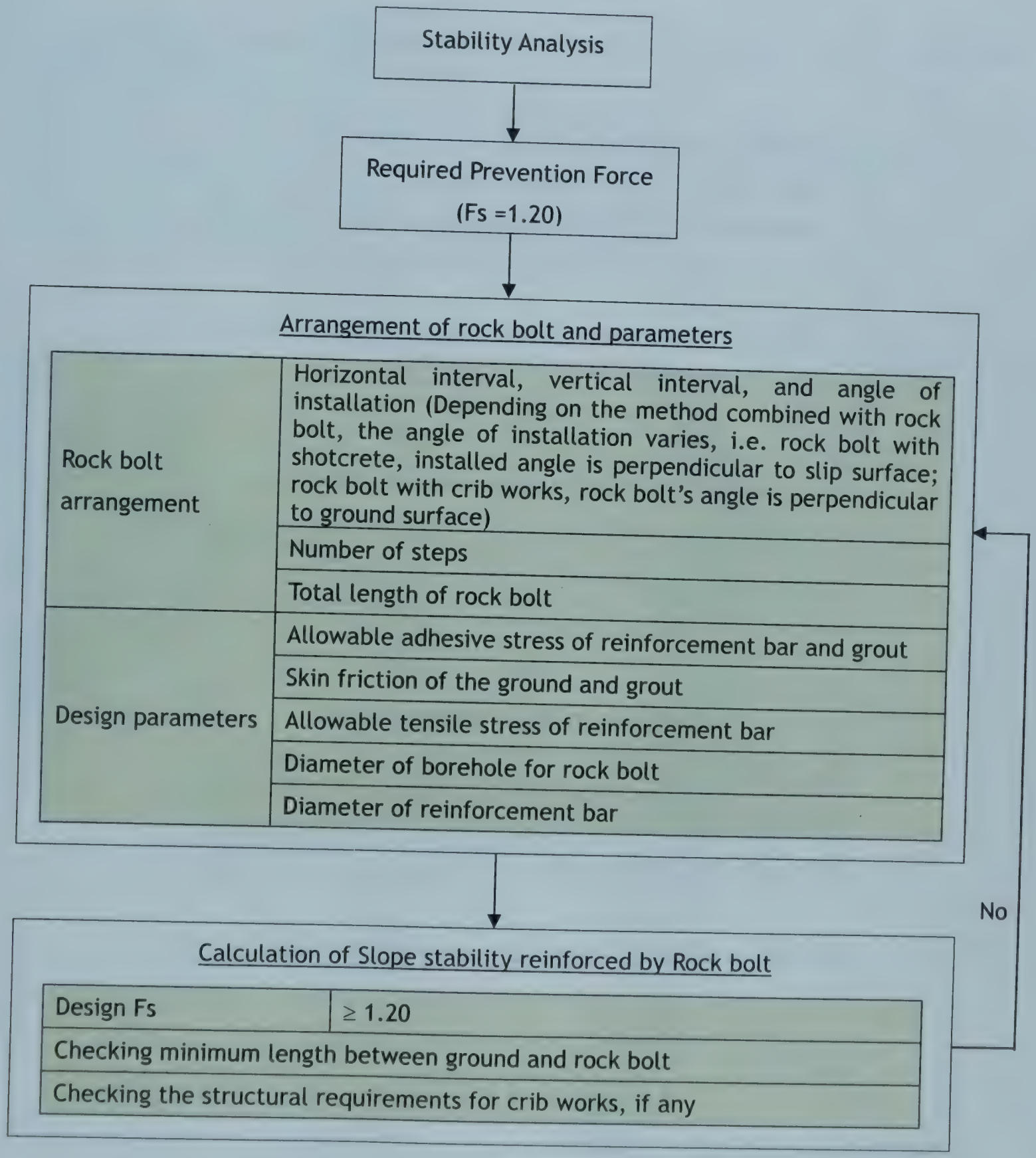


Figure 4-3 Design procedure of Rock bolt

4.3 Procedure for Concrete Crib Works

Procedure for quantity estimation and arrangement of frames of concrete crib works.

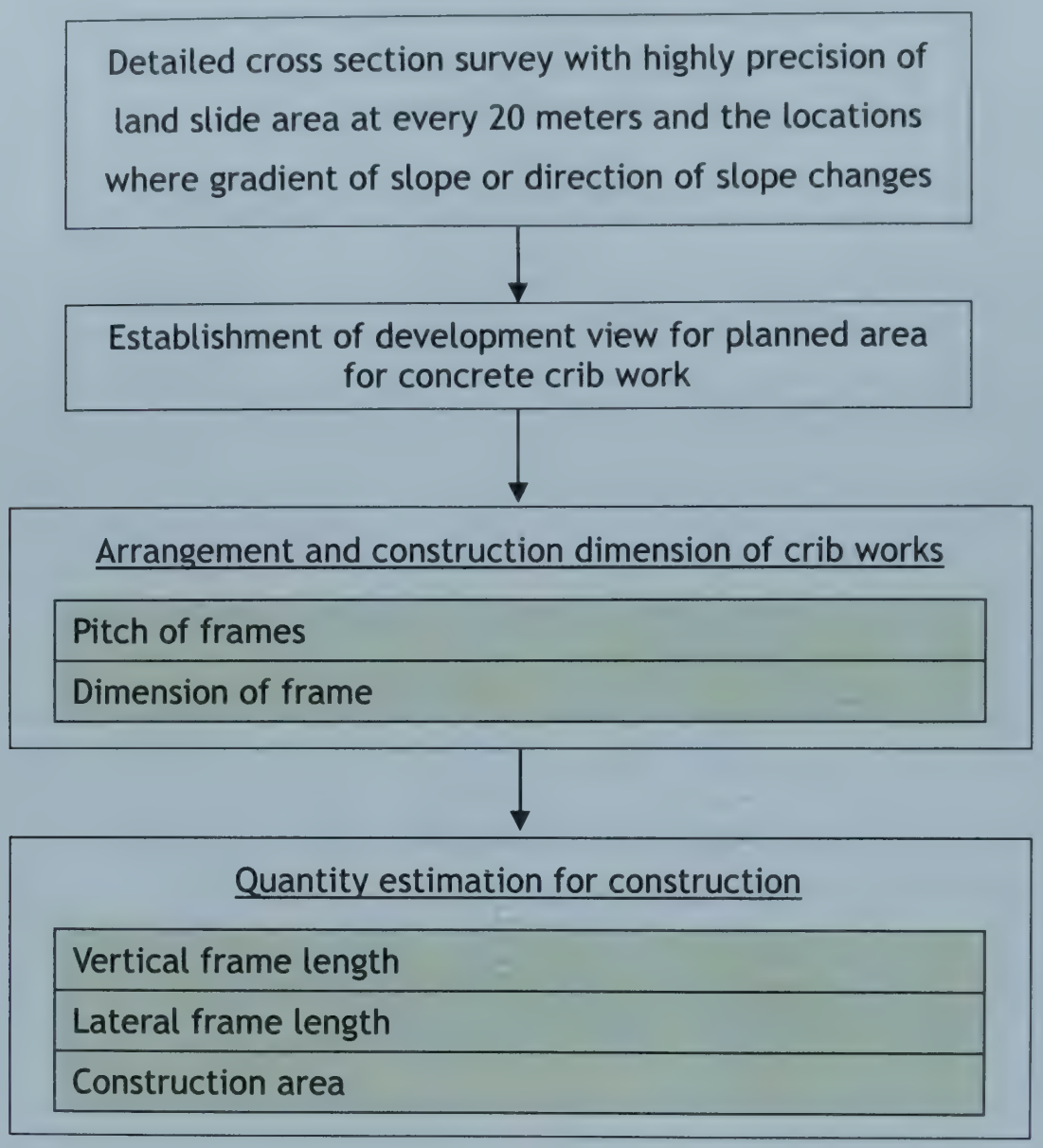
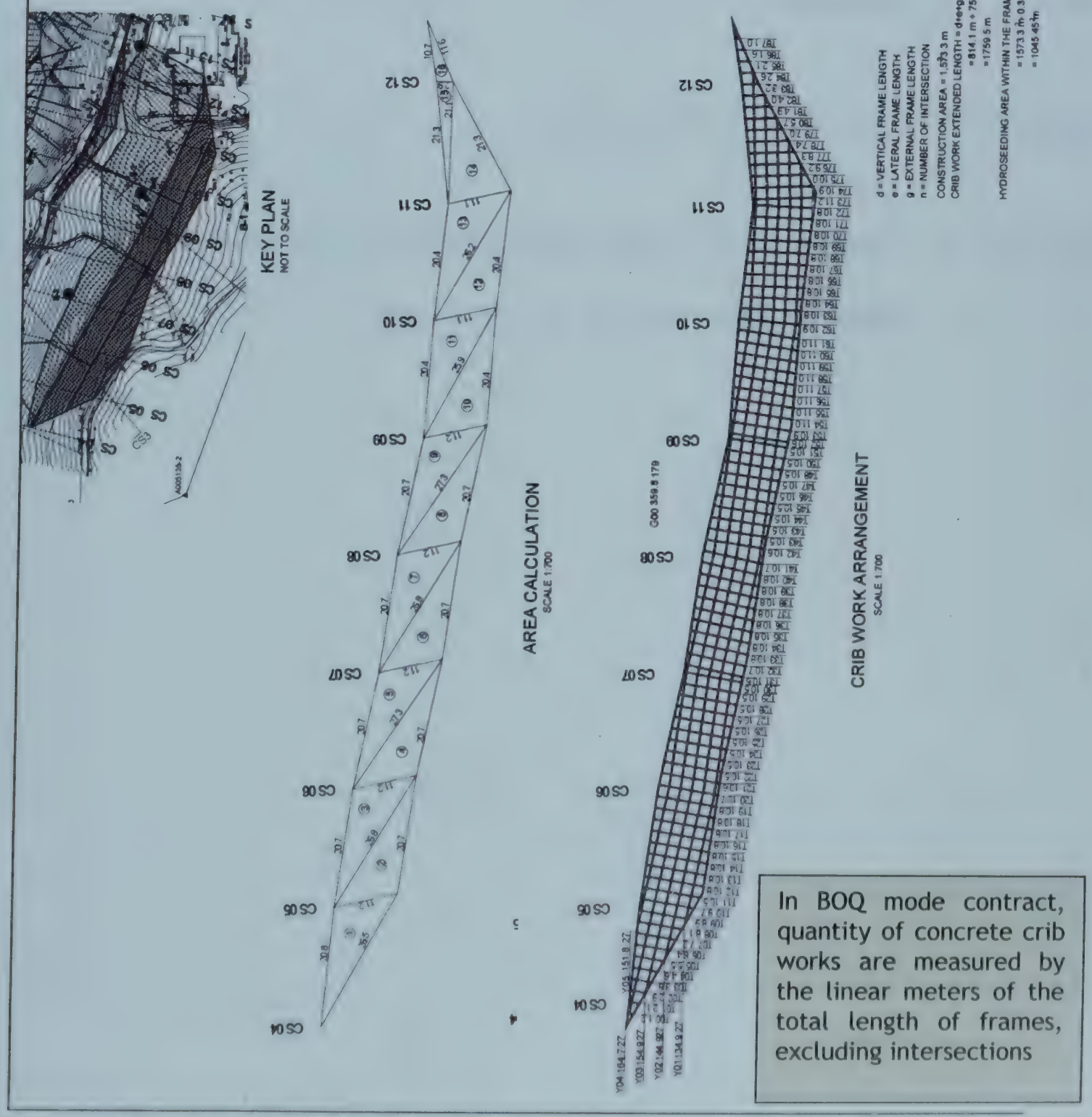


Figure 4-4 Procedure of quantity estimation for crib works



In BOQ mode contract, quantity of concrete crib works are measured by the linear meters of the total length of frames, excluding intersections

Figure 4-5 Area Calculation and Arrangement for Concrete Crib Works

Design for Slide No.1 and No.2

Appendix 1.1 Back Analysis Results

Appendix 1.2 Calculation of Rock Bolt

Appendix 1.3 Calculation of Crib Works

Appendix 1.4 Drawings (Plan View, Representative Cross sections and Representative drawing of crib works)

Appendix 1.1

Back Analysis Results

Back Analysis Result

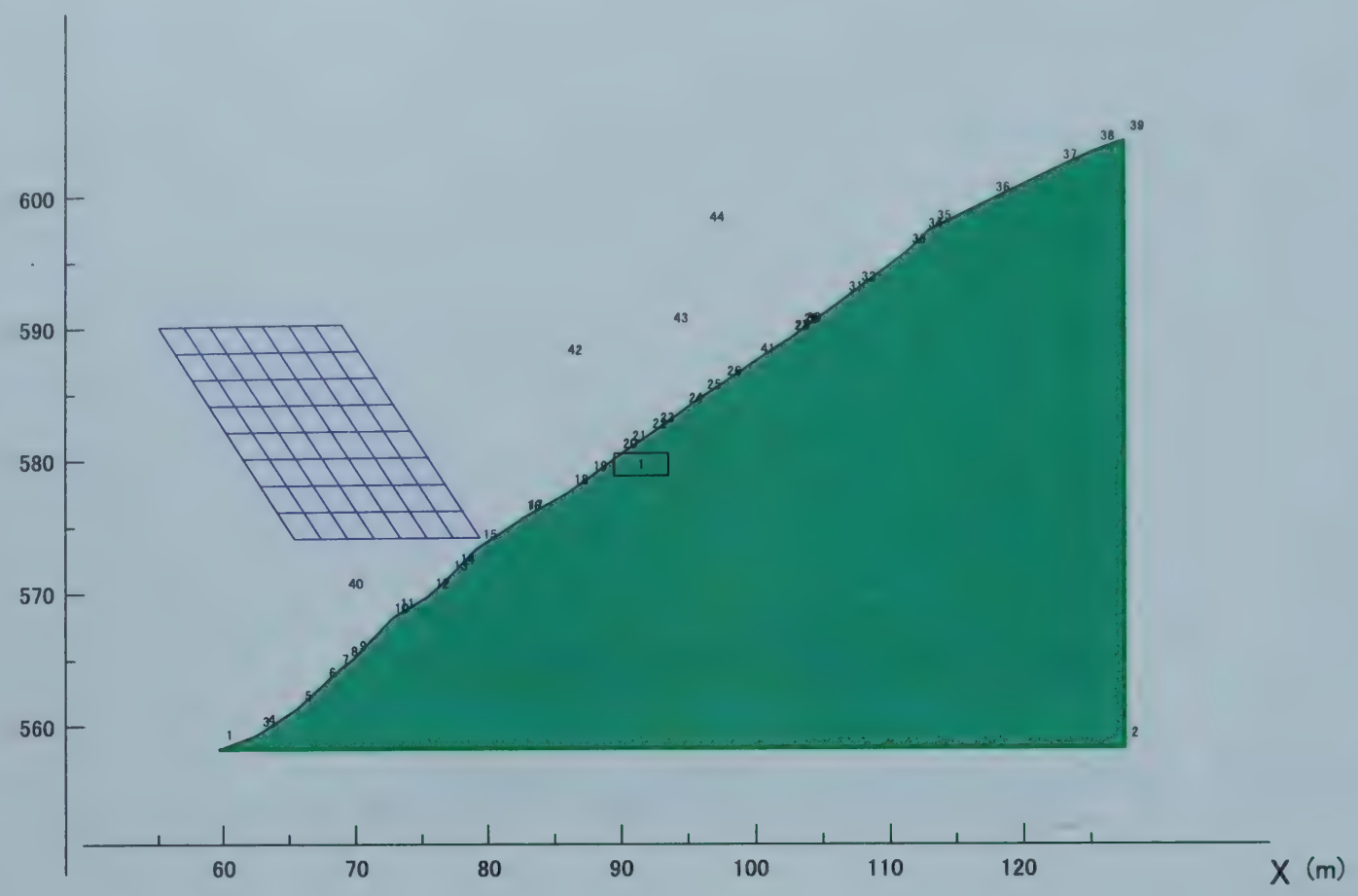
Slide 1 (Zone 1, Zone 2, and Zone 4-1)

Back Analysis Result

Slide 1-Slip Surface 1 (Zone 1)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....

Analysis type ..... Inverse analysis

Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{R\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{R\sum W*\sin(a)}$$

Fs : Safety factor

R : Radius (m)

C : Cohesion (kN/m²)

L : Circle length of a slice (m)

W : Total weight of a slice (kN/m)

a : Angle (degree)

U : Pore water pressure (kN/m²)

b : Width of a slice (m)

PHI : Friction angle (degree)

(DATA 02.) // CONTROL //

Req. for safety factor (Ordinary) 1.000

Req. for safety factor (Seismic) 1.000

(DATA 08.) // LAYER //

No.	material property No.	Num of node	Node numbers composing a layer										
1	1	40	1	2	39	38	37	36	35	34	33	32	31
			30	29	28	27	41	26	25	24	23	22	21
			20	19	18	17	16	15	14	13	12	11	10
			9	8	7	6	5	4	3				

(DATA 09.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Unit weight for calculation of pore water (kN/m^3)	Alpha	Cohesion (kN/m^2)	rate of increase of Cohesion	Friction angle (degree)	TAU (kN/m^2)
1	18.00	18.00	----	----	----	----	----	----

The layer for Inverse analysis

(DATA 11.) // GRID POINT //

No.	Slip direction	Double cutting	GX (m)	GY (m)	X-point	Y-point	X-pitch (m)	Y-pitch (m)	Dx/Dy
1	UPSTREAM	RIGHT	55.00	590.00	8	9	2.00	2.00	0.650

(DATA 12.) // RADIUS // (Perpendicular)

Radius ..... From up to down

Pitch (m) 0.25

Min. Radius (Y-coordinate) (m)..... 586.91

Max. Radius (Y-coordinate) (m)..... 569.59

(DATA 16.) // NOT CUT LINE // (N = 2)

Num of Points	X (m)	Y (m)
2	127.41	558.25
	127.41	603.23
2	127.41	558.25
	59.66	558.25

2. Inverse Analysis

Inverse ANALYSIS (Group number = 1)

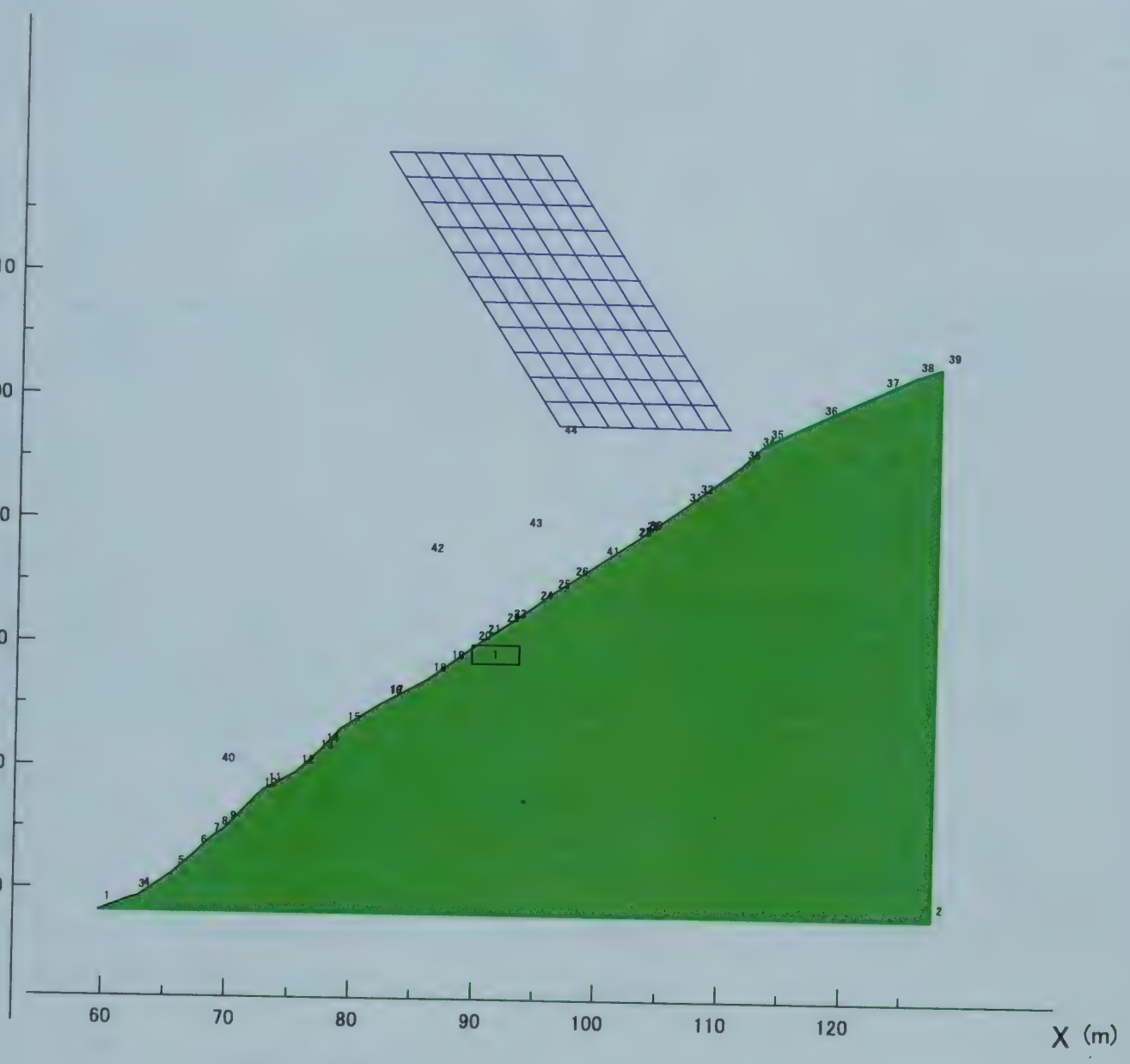
Ordinary condition					
C (kN/m²)	PH (degree)	tan(PH)	PH (degree)	tan(PH)	C (kN/m²)
0.0	46.60	1.06	0.00	0.00	16.02
1.0	36.02	0.73	1.00	0.02	15.54
2.0	30.52	0.59	2.00	0.03	15.06
3.0	27.65	0.52	3.00	0.05	14.58
4.0	24.98	0.47	4.00	0.07	14.10
5.0	22.88	0.42	5.00	0.09	13.62
6.0	20.86	0.38	6.00	0.11	13.14
7.0	18.79	0.34	7.00	0.12	12.65
8.0	16.66	0.30	8.00	0.14	12.16
9.0	14.49	0.26	9.00	0.16	11.67
10.0	12.37	0.22	10.00	0.18	11.18
11.0	10.37	0.18	11.00	0.19	10.69
12.0	8.34	0.15	12.00	0.21	10.19
13.0	6.28	0.11	13.00	0.23	9.69
14.0	4.21	0.07	14.00	0.25	9.22
15.0	2.13	0.04	15.00	0.27	8.77
16.0	0.05	0.00	16.00	0.29	8.31
17.0	-2.04	-0.04	17.00	0.31	7.84
18.0	-3.94	-0.07	18.00	0.32	7.37
19.0	-5.77	-0.10	19.00	0.34	6.90
20.0	-7.59	-0.13	20.00	0.36	6.42
21.0	-9.40	-0.17	21.00	0.38	5.93
22.0	-11.19	-0.20	22.00	0.40	5.44
23.0	-12.95	-0.23	23.00	0.42	4.94
24.0	-14.69	-0.26	24.00	0.45	4.44
25.0	-16.17	-0.29	25.00	0.47	3.99
26.0	-17.58	-0.32	26.00	0.49	3.59
27.0	-18.97	-0.34	27.00	0.51	3.23
28.0	-20.34	-0.37	28.00	0.53	2.87
29.0	-21.68	-0.40	29.00	0.55	2.51
30.0	-23.00	-0.42	30.00	0.58	2.15
31.0	-24.29	-0.45	31.00	0.60	1.92
32.0	-25.56	-0.48	32.00	0.62	1.74
33.0	-26.80	-0.51	33.00	0.65	1.56
34.0	-28.02	-0.53	34.00	0.67	1.38
35.0	-29.21	-0.56	35.00	0.70	1.19
36.0	-30.37	-0.59	36.00	0.73	1.00
37.0	-31.50	-0.61	37.00	0.75	0.82
38.0	-32.61	-0.64	38.00	0.78	0.68
39.0	-33.69	-0.67	39.00	0.81	0.55
40.0	-34.75	-0.69	40.00	0.84	0.42
41.0	-35.78	-0.72	41.00	0.87	0.32
42.0	-36.78	-0.75	42.00	0.90	0.25
43.0	-37.75	-0.77	43.00	0.93	0.17
44.0	-38.71	-0.80	44.00	0.97	0.10
45.0	-39.63	-0.83	45.00	1.00	0.03
46.0	-40.54	-0.86	46.00	1.04	0.00
47.0	-41.41	-0.88	47.00	1.07	0.00
48.0	-42.27	-0.91	48.00	1.11	-0.01
49.0	-43.10	-0.94	49.00	1.15	-0.01
50.0	-43.92	-0.96	50.00	1.19	-0.02

Back Analysis Result

Slide 1-Slip Surface 2 (Zone 1)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....
Analysis type ..... Inverse analysis
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{R\Sigma[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{R\Sigma W*\sin(a)}$$

Fs : Safety factor
R : Radius (m)
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

(DATA 02.) // CONTROL //

Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000

(DATA 08.) // LAYER //

No.	material property No.	Num of node	Node numbers composing a layer									
1	1	40	1	2	39	38	37	36	35	34	33	32	31
			30	29	28	27	41	26	25	24	23	22	21
			20	19	18	17	16	15	14	13	12	11	10
			9	8	7	6	5	4	3				

(DATA 09.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Unit weight for calculation of pore water (kN/m^3)	Alpha	Cohesion (kN/m^2)	rate of increase of Cohesion	Friction angle (degree)	TAU (kN/m^2)
1	18.00	18.00	----	----	----	----	----	----

The layer for Inverse analysis

(DATA 11.) // GRID POINT //

No.	Slip direction	Double cutting	GX (m)	GY (m)	X-point	Y-point	X-pitch (m)	Y-pitch (m)	Dx/Dy
1	UPSTREAM	RIGHT	82.00	620.00	8	12	2.00	2.00	0.650

(DATA 12.) // RADIUS // (Perpendiculer)

Radius ..... From up to down

Pitch (m) 0.25

Min. Radius (Y-coordinate) (m)..... 596.72

Max. Radius (Y-coordinate) (m)..... 589.18

(DATA 16.) // NOT CUT LINE // (N = 3)

Num of Points	X (m)	Y (m)
2	127.41	558.25
	127.41	603.23
2	127.41	558.25
	59.66	558.25
5	127.41	603.23
	125.21	602.46
	122.41	601.14
	117.41	598.78
	113.04	596.72

2. Inverse Analysis

Inverse ANALYSIS (Group number = 1)

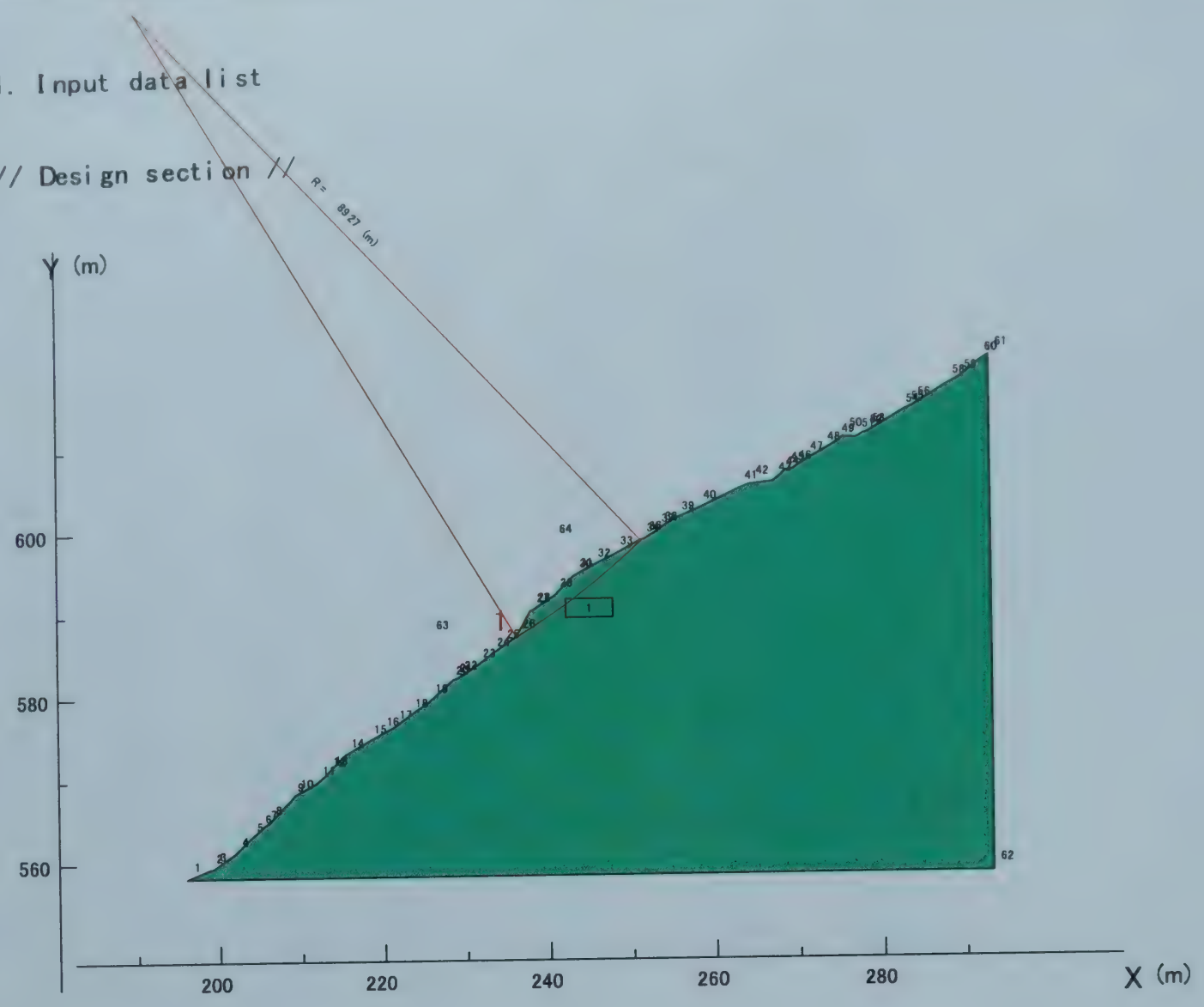
Ordinary condition					
C (kN/m²)	PHI (degree)	tan(PHI)	PHI (degree)	tan(PHI)	C (kN/m²)
0.0	43.36	0.94	0.00	0.00	12.87
1.0	31.64	0.62	1.00	0.02	12.52
2.0	28.70	0.55	2.00	0.03	12.16
3.0	25.91	0.49	3.00	0.05	11.81
4.0	23.59	0.44	4.00	0.07	11.45
5.0	21.18	0.39	5.00	0.09	11.09
6.0	18.68	0.34	6.00	0.11	10.73
7.0	16.12	0.29	7.00	0.12	10.38
8.0	13.48	0.24	8.00	0.14	10.01
9.0	10.78	0.19	9.00	0.16	9.65
10.0	8.04	0.14	10.00	0.18	9.29
11.0	5.26	0.09	11.00	0.19	8.92
12.0	2.45	0.04	12.00	0.21	8.55
13.0	-0.37	-0.01	13.00	0.23	8.18
14.0	-3.18	-0.06	14.00	0.25	7.80
15.0	-5.99	-0.10	15.00	0.27	7.43
16.0	-8.76	-0.15	16.00	0.29	7.04
17.0	-11.49	-0.20	17.00	0.31	6.66
18.0	-14.17	-0.25	18.00	0.32	6.27
19.0	-16.79	-0.30	19.00	0.34	5.87
20.0	-19.34	-0.35	20.00	0.36	5.48
21.0	-21.81	-0.40	21.00	0.38	5.07
22.0	-24.20	-0.45	22.00	0.40	4.66
23.0	-26.50	-0.50	23.00	0.42	4.25
24.0	-28.72	-0.55	24.00	0.45	3.83
25.0	-30.84	-0.60	25.00	0.47	3.40
26.0	-32.88	-0.65	26.00	0.49	2.96
27.0	-34.82	-0.70	27.00	0.51	2.55
28.0	-36.68	-0.74	28.00	0.53	2.23
29.0	-38.45	-0.79	29.00	0.55	1.90
30.0	-40.14	-0.84	30.00	0.58	1.56
31.0	-41.75	-0.89	31.00	0.60	1.22
32.0	-43.28	-0.94	32.00	0.62	0.87
33.0	-44.74	-0.99	33.00	0.65	0.63
34.0	-46.13	-1.04	34.00	0.67	0.43
35.0	-47.45	-1.09	35.00	0.70	0.27
36.0	-48.71	-1.14	36.00	0.73	0.20
37.0	-49.91	-1.19	37.00	0.75	0.14
38.0	-51.05	-1.24	38.00	0.78	0.10
39.0	-52.14	-1.29	39.00	0.81	0.06
40.0	-53.18	-1.34	40.00	0.84	0.04
41.0	-54.16	-1.38	41.00	0.87	0.03
42.0	-55.11	-1.43	42.00	0.90	0.02
43.0	-56.01	-1.48	43.00	0.93	0.00
44.0	-56.87	-1.53	44.00	0.97	-0.01
45.0	-57.70	-1.58	45.00	1.00	-0.02
46.0	-58.48	-1.63	46.00	1.04	-0.03
47.0	-59.24	-1.68	47.00	1.07	-0.05
48.0	-59.96	-1.73	48.00	1.11	-0.06
49.0	-60.65	-1.78	49.00	1.15	-0.08
50.0	-61.32	-1.83	50.00	1.19	-0.09

Back Analysis Result

Slide 1-Slip Surface 3 (Zone 4-1)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....
Analysis type ..... Inverse analysis
Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{\sum W*\sin(a)}$$

Fs : Safety factor
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

Width of a slice 10.0
Load condition ..... 0 (= 0 Affects both resisting and sliding forces)
(= 1 Only sliding force)

(DATA 03.) // NODAL POINT //

No.	Coordinate	
	X (m)	Y (m)
45	268.77	606.27
46	269.60	606.46
47	271.10	607.46
48	273.13	608.59
49	274.84	609.54
50	275.68	610.18
51	277.20	610.08
52	278.13	610.53
53	278.47	610.70
54	282.55	612.96
55	283.13	613.29
56	283.96	613.75
58	288.15	616.26
59	289.53	616.92
60	292.00	618.89
61	293.13	619.60
62	293.13	558.25
63	225.80	586.83
64	240.83	598.14

(DATA 05.) // LAYER //

No.	Material property No.	Num of node	Node numbers composing a layer									
1	1	61	1	62	61	60	59	58	56	55	54	53	52	51
			50	49	48	47	46	45	44	43	42	41	40	39
			38	37	36	35	34	33	32	31	30	29	28	27
			26	25	24	23	22	21	20	19	18	17	16	15
			14	13	12	11	10	9	8	7	6	5	4	3
			2											

(DATA 06.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Cohesion (kN/m^2)	rate of increase of cohesion	Friction angle (Degree)	Tau (kN/m^2)
1	18.00	18.00	------	-------	--------	-----

The layer for Inverse analysis 1

Y-Coordinate of base line of cohesion (m) = 0.00

3. Inverse Analysis

INVERSE ANALYSIS LIST

Ordinary condition					
Cohesion (kN/m²)	Angle (Degree)	tan(PHI)	Angle (Degree)	tan(PHI)	Cohesion (kN/m²)
0.0	36.15	0.73	0.00	0.00	13.79
1.0	34.13	0.68	1.00	0.02	13.46
2.0	31.99	0.62	2.00	0.03	13.13
3.0	29.76	0.57	3.00	0.05	12.80
4.0	27.42	0.52	4.00	0.07	12.47
5.0	24.98	0.47	5.00	0.09	12.14
6.0	22.43	0.41	6.00	0.11	11.81
7.0	19.79	0.36	7.00	0.12	11.48
8.0	17.06	0.31	8.00	0.14	11.14
9.0	14.25	0.25	9.00	0.16	10.80
10.0	11.36	0.20	10.00	0.18	10.47
11.0	8.42	0.15	11.00	0.19	10.12
12.0	5.43	0.10	12.00	0.21	9.78
13.0	2.41	0.04	13.00	0.23	9.44
14.0	0.00	0.00	14.00	0.25	9.09
15.0	0.00	0.00	15.00	0.27	8.74
16.0	0.00	0.00	16.00	0.29	8.38
17.0	0.00	0.00	17.00	0.31	8.02
18.0	0.00	0.00	18.00	0.32	7.66
19.0	0.00	0.00	19.00	0.34	7.29
20.0	0.00	0.00	20.00	0.36	6.92
21.0	0.00	0.00	21.00	0.38	6.55
22.0	0.00	0.00	22.00	0.40	6.17
23.0	0.00	0.00	23.00	0.42	5.78
24.0	0.00	0.00	24.00	0.45	5.39
25.0	0.00	0.00	25.00	0.47	4.99
26.0	0.00	0.00	26.00	0.49	4.59
27.0	0.00	0.00	27.00	0.51	4.17
28.0	0.00	0.00	28.00	0.53	3.76
29.0	0.00	0.00	29.00	0.55	3.33
30.0	0.00	0.00	30.00	0.58	2.89
31.0	0.00	0.00	31.00	0.60	2.45
32.0	0.00	0.00	32.00	0.62	2.00
33.0	0.00	0.00	33.00	0.65	1.53
34.0	0.00	0.00	34.00	0.67	1.06
35.0	0.00	0.00	35.00	0.70	0.58
36.0	0.00	0.00	36.00	0.73	0.08
37.0	0.00	0.00	37.00	0.75	0.00
38.0	0.00	0.00	38.00	0.78	0.00
39.0	0.00	0.00	39.00	0.81	0.00
40.0	0.00	0.00	40.00	0.84	0.00
41.0	0.00	0.00	41.00	0.87	0.00
42.0	0.00	0.00	42.00	0.90	0.00
43.0	0.00	0.00	43.00	0.93	0.00
44.0	0.00	0.00	44.00	0.97	0.00
45.0	0.00	0.00	45.00	1.00	0.00
46.0	0.00	0.00	46.00	1.04	0.00
47.0	0.00	0.00	47.00	1.07	0.00
48.0	0.00	0.00	48.00	1.11	0.00
49.0	0.00	0.00	49.00	1.15	0.00
50.0	0.00	0.00	50.00	1.19	0.00

Back Analysis Result

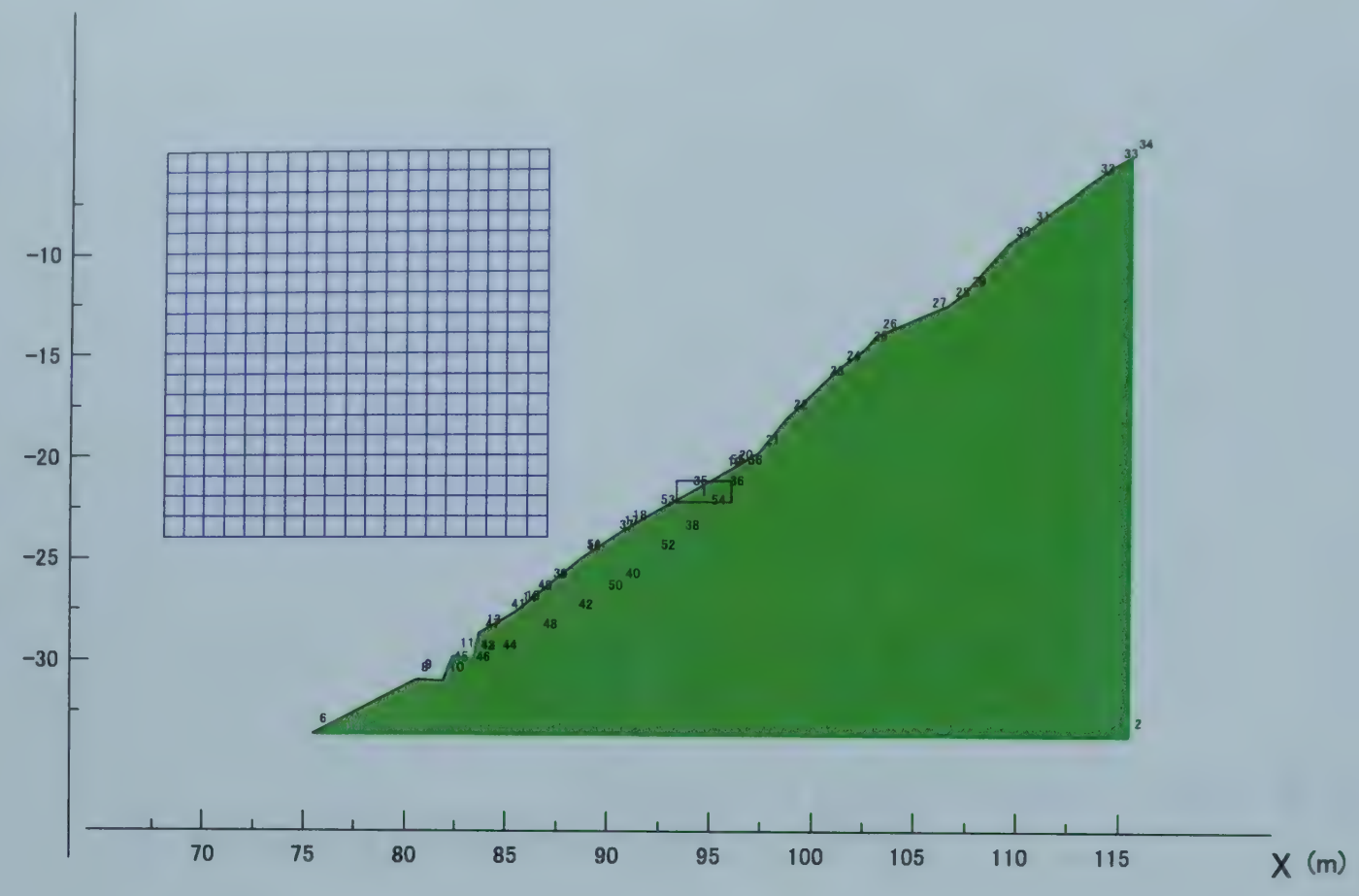
Slide 2 (Zone 3 and Zone 4-2)

Back Analysis Result

Slide 2-Slip Surface 1 (Zone 3)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title ..... CS 06 - Slope Stability
Analysis type ..... Inverse analysis
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{R\tilde{}[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{R\tilde{}W*\sin(a)}$$

Fs : Safety factor
R : Radius (m)
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

(DATA 02.) // CONTROL //

Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000

(DATA 06.) // NODAL POINT //

No.	Coordinate		Y-coordinate before settlement (m)
	x (m)	y (m)	
50	89.68	-27.05	0.00
51	88.63	-25.09	0.00
52	92.28	-25.09	0.00
53	92.20	-22.86	0.00
54	94.72	-22.86	0.00
55	95.58	-20.92	0.00
56	96.50	-20.92	0.00

(DATA 08.) // LAYER //

No.	material property No.	Num of node	Node numbers composing a layer									
1	1	39	6	2	34	33	32	31	30	29	28	27	26
			25	24	23	22	21	20	55	19	35	53	18
			17	37	51	16	39	49	15	14	41	13	47
			12	11	45	10	9	8					

(DATA 09.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m³)	Wet weight (kN/m³)	Unit weight for calculation of pore water (kN/m³)	Alpha	Cohesion (kN/m²)	rate of increase of Cohesion	Friction angle (degree)	TAU (kN/m²)
1	18.00	18.00	----	----	----	----	----	----

The layer for Inverse analysis

(DATA 11.) // GRID POINT //

No.	Slip direction	Double cutting	GX (m)	GY (m)	X-point	Y-point	X-pitch (m)	Y-pitch (m)	Dx/Dy
1	UPSTREAM	RIGHT	68.00	-5.00	20	20	1.00	1.00	0.000

(DATA 12.) // RADIUS // (Perpendiculer)

Radius ..... From down to up

Pitch (m) 0.50

Min. Radius (Y-coordinate) (m)..... -12.19

Max. Radius (Y-coordinate) (m)..... -31.05

(DATA 16.) // NOT CUT LINE // (N = 3)

Num of Points	X (m)	Y (m)
2	115.43	-33.70
	115.43	-5.50
2	115.43	-33.70

2. Inverse Analysis

Inverse ANALYSIS (Group number = 1)

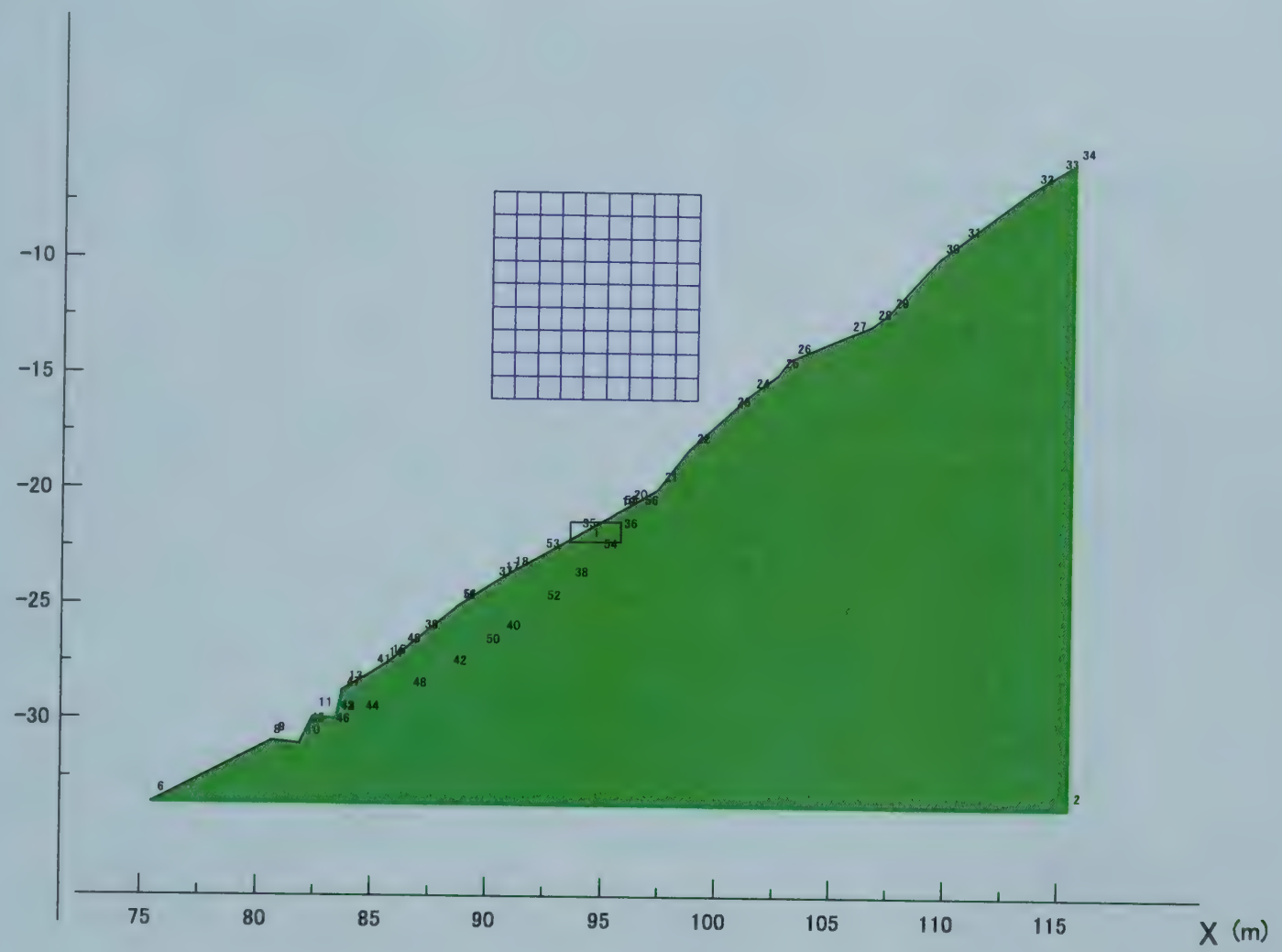
Ordinary condition					
C (kN/m²)	PHI (degree)	tan(PHI)	PHI (degree)	tan(PHI)	C (kN/m²)
0.0	53.50	1.35	0.00	0.00	45.46
1.0	35.13	0.70	1.00	0.02	44.11
2.0	31.81	0.62	2.00	0.03	42.76
3.0	30.57	0.59	3.00	0.05	41.41
4.0	29.57	0.57	4.00	0.07	40.06
5.0	28.66	0.55	5.00	0.09	38.71
6.0	27.74	0.53	6.00	0.11	37.35
7.0	26.85	0.51	7.00	0.12	35.98
8.0	26.02	0.49	8.00	0.14	34.61
9.0	25.35	0.47	9.00	0.16	33.24
10.0	24.70	0.46	10.00	0.18	31.85
11.0	24.06	0.45	11.00	0.19	30.46
12.0	23.44	0.43	12.00	0.21	29.06
13.0	22.81	0.42	13.00	0.23	27.64
14.0	22.18	0.41	14.00	0.25	26.22
15.0	21.54	0.39	15.00	0.27	24.78
16.0	20.89	0.38	16.00	0.29	23.33
17.0	20.24	0.37	17.00	0.31	21.87
18.0	19.59	0.36	18.00	0.32	20.38
19.0	18.92	0.34	19.00	0.34	18.89
20.0	18.26	0.33	20.00	0.36	17.37
21.0	17.59	0.32	21.00	0.38	15.84
22.0	16.91	0.30	22.00	0.40	14.28
23.0	16.23	0.29	23.00	0.42	12.70
24.0	15.54	0.28	24.00	0.45	11.10
25.0	14.85	0.27	25.00	0.47	9.54
26.0	14.15	0.25	26.00	0.49	8.03
27.0	13.45	0.24	27.00	0.51	6.82
28.0	12.75	0.23	28.00	0.53	5.72
29.0	12.04	0.21	29.00	0.55	4.63
30.0	11.33	0.20	30.00	0.58	3.56
31.0	10.61	0.19	31.00	0.60	2.57
32.0	9.89	0.17	32.00	0.62	1.92
33.0	9.17	0.16	33.00	0.65	1.52
34.0	8.45	0.15	34.00	0.67	1.21
35.0	7.72	0.14	35.00	0.70	1.01
36.0	6.99	0.12	36.00	0.73	0.92
37.0	6.25	0.11	37.00	0.75	0.82
38.0	5.52	0.10	38.00	0.78	0.72
39.0	4.78	0.08	39.00	0.81	0.62
40.0	4.05	0.07	40.00	0.84	0.51
41.0	3.31	0.06	41.00	0.87	0.40
42.0	2.57	0.04	42.00	0.90	0.29
43.0	1.82	0.03	43.00	0.93	0.23
44.0	1.08	0.02	44.00	0.97	0.19
45.0	0.34	0.01	45.00	1.00	0.14
46.0	-0.40	-0.01	46.00	1.04	0.11
47.0	-1.15	-0.02	47.00	1.07	0.08
48.0	-1.89	-0.03	48.00	1.11	0.06
49.0	-2.63	-0.05	49.00	1.15	0.04
50.0	-3.37	-0.06	50.00	1.19	0.01

Back Analysis Result

Slide 2-Slip Surface 2 (Zone 4-2)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title ..... CS 06 - Slope Stability
Analysis type ..... Inverse analysis
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{R\Sigma[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{R\Sigma W*\sin(a)}$$

Fs : Safety factor
R : Radius (m)
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

(DATA 02.) // CONTROL //
Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000

(DATA 06.) // NODAL POINT //

No.	Coordinate		Y-coordinate
	x (m)	y (m)	before settlement (m)
50	89.68	-27.05	0.00
51	88.63	-25.09	0.00
52	92.28	-25.09	0.00
53	92.20	-22.86	0.00
54	94.72	-22.86	0.00
55	95.58	-20.92	0.00
56	96.50	-20.92	0.00

(DATA 08.) // LAYER //

No.	material property No.	Num of node	Node numbers composing a layer									
1	1	39	6	2	34	33	32	31	30	29	28	27	26
			25	24	23	22	21	20	55	19	35	53	18
			17	37	51	16	39	49	15	14	41	13	47
			12	11	45	10	9	8					

(DATA 09.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Unit weight for calculation of pore water (kN/m^3)	Alpha	Cohesion (kN/m^2)	rate of increase of Cohesion	Friction angle (degree)	TAU (kN/m^2)
1	18.00	18.00	----	----	----	----	----	----

The layer for Inverse analysis

(DATA 11.) // GRID POINT //

No.	Slip direction	Double cutting	GX (m)	GY (m)	X-point	Y-point	X-pitch (m)	Y-pitch (m)	Dx/Dy
1	UPSTREAM	RIGHT	90.00	-7.00	10	10	1.00	1.00	0.000

(DATA 12.) // RADIUS // (Perpendiculer)

Radius ..... From up to down

Pitch (m) 0.25

Min. Radius (Y-coordinate) (m)..... -12.18

Max. Radius (Y-coordinate) (m)..... -19.90

(DATA 16.) // NOT CUT LINE // (N = 3)

Num of Points	X (m)	Y (m)
2	115.43	-33.70
	115.43	-5.50
2	115.43	-33.70

2. Inverse Analysis

Inverse ANALYSIS (Group number = 1)

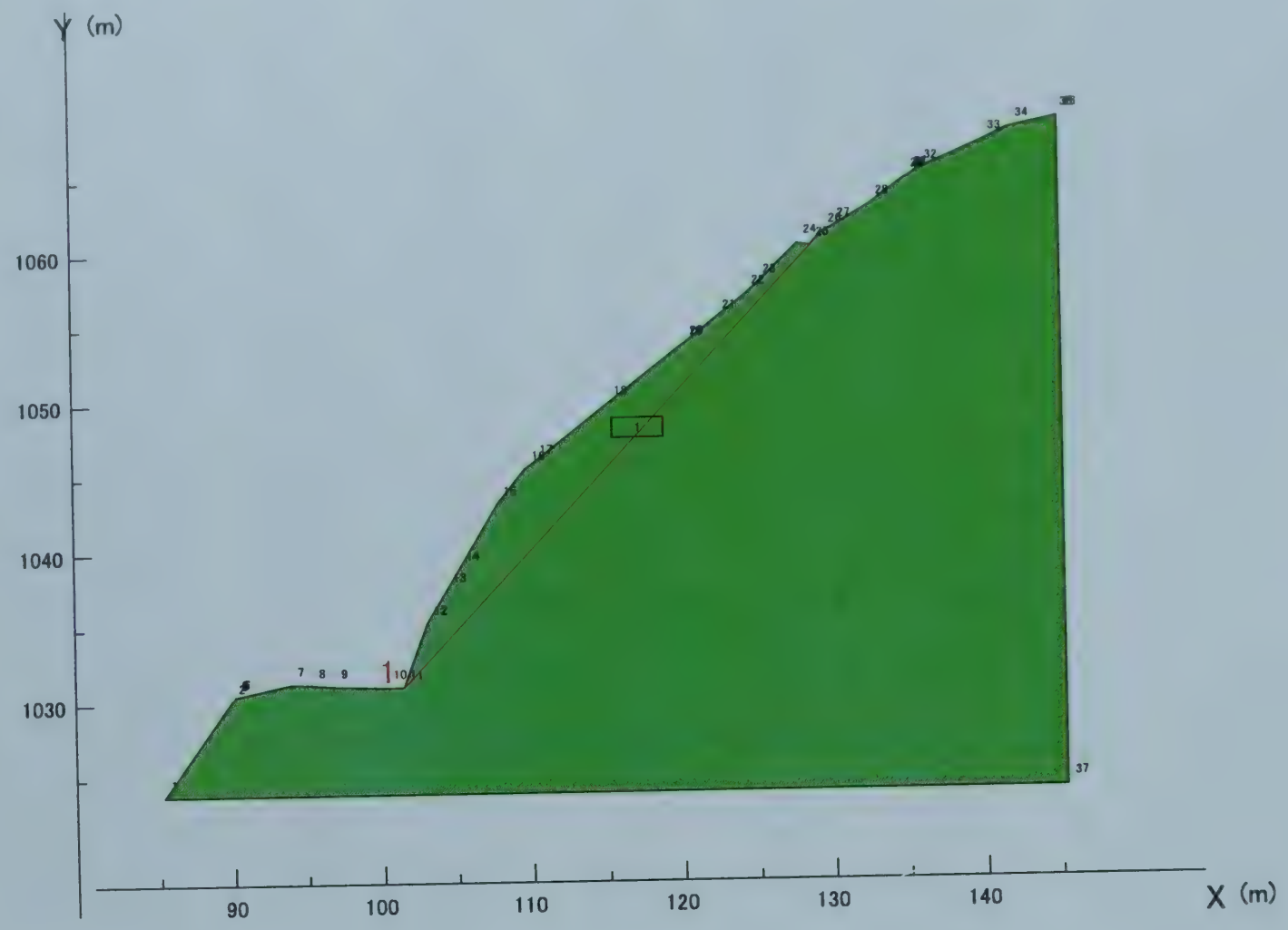
Ordinary condition					
C (kN/m²)	PHI (degree)	tan(PHI)	PHI (degree)	tan(PHI)	C (kN/m²)
0.0	51.43	1.25	0.00	0.00	23.41
1.0	36.93	0.75	1.00	0.02	22.73
2.0	33.17	0.65	2.00	0.03	22.06
3.0	30.14	0.58	3.00	0.05	21.38
4.0	27.47	0.52	4.00	0.07	20.70
5.0	25.44	0.48	5.00	0.09	20.02
6.0	24.22	0.45	6.00	0.11	19.34
7.0	22.98	0.42	7.00	0.12	18.66
8.0	21.71	0.40	8.00	0.14	17.97
9.0	20.42	0.37	9.00	0.16	17.28
10.0	19.11	0.35	10.00	0.18	16.58
11.0	17.78	0.32	11.00	0.19	15.89
12.0	16.43	0.29	12.00	0.21	15.18
13.0	15.06	0.27	13.00	0.23	14.47
14.0	13.67	0.24	14.00	0.25	13.76
15.0	12.26	0.22	15.00	0.27	13.04
16.0	10.84	0.19	16.00	0.29	12.31
17.0	9.40	0.17	17.00	0.31	11.58
18.0	7.96	0.14	18.00	0.32	10.84
19.0	6.50	0.11	19.00	0.34	10.09
20.0	5.03	0.09	20.00	0.36	9.33
21.0	3.56	0.06	21.00	0.38	8.56
22.0	2.08	0.04	22.00	0.40	7.78
23.0	0.60	0.01	23.00	0.42	6.98
24.0	-0.88	-0.02	24.00	0.45	6.18
25.0	-2.36	-0.04	25.00	0.47	5.37
26.0	-3.84	-0.07	26.00	0.49	4.59
27.0	-5.31	-0.09	27.00	0.51	4.17
28.0	-6.77	-0.12	28.00	0.53	3.81
29.0	-8.23	-0.14	29.00	0.55	3.43
30.0	-9.67	-0.17	30.00	0.58	3.06
31.0	-11.11	-0.20	31.00	0.60	2.72
32.0	-12.52	-0.22	32.00	0.62	2.39
33.0	-13.93	-0.25	33.00	0.65	2.06
34.0	-15.31	-0.27	34.00	0.67	1.72
35.0	-16.68	-0.30	35.00	0.70	1.44
36.0	-18.03	-0.33	36.00	0.73	1.21
37.0	-19.36	-0.35	37.00	0.75	0.98
38.0	-20.67	-0.38	38.00	0.78	0.81
39.0	-21.95	-0.40	39.00	0.81	0.65
40.0	-23.22	-0.43	40.00	0.84	0.56
41.0	-24.46	-0.45	41.00	0.87	0.46
42.0	-25.67	-0.48	42.00	0.90	0.36
43.0	-26.86	-0.51	43.00	0.93	0.25
44.0	-28.03	-0.53	44.00	0.97	0.16
45.0	-29.17	-0.56	45.00	1.00	0.11
46.0	-30.29	-0.58	46.00	1.04	0.05
47.0	-31.38	-0.61	47.00	1.07	0.01
48.0	-32.45	-0.64	48.00	1.11	0.00
49.0	-33.49	-0.66	49.00	1.15	0.00
50.0	-34.51	-0.69	50.00	1.19	0.00

Back Analysis Result

Loosen Slope A (Zone 7)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....
Analysis type ..... Inverse analysis
Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{\sum W*\sin(a)}$$

Fs : Safety factor
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

Width of a slice 10.0
Load condition ..... 0 (= 0 Affects both resisting and sliding forces)
(= 1 Only sliding force)

(DATA 05.) // LAYER //

No.	Material property No.	Num of node	Node numbers composing a layer											
1	1	36	1	37	36	35	34	33	32	31	30	29	28	27
			26	25	24	23	22	21	20	19	18	17	16	15
			14	13	12	11	10	9	8	7	6	5	3	2

(DATA 06.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Cohesion (kN/m^2)	rate of increase of cohesion	Friction angle (Degree)	Tau (kN/m^2)
1	18.00	18.00	------	-------	--------	-----

The layer for Inverse analysis 1

Y-Coordinate of base line of cohesion (m) = 0.00

(DATA 07.) // SLIP SURFACE //

Slip surface No. = 1

NODE NO.	1	2
X (m)	101.41	128.87
Y (m)	1030.89	1059.64
R (m)	0.00	0.00
FIXED X (m)	0.00	0.00
FIXED Y (m)	0.00	0.00

3. Inverse Analysis

INVERSE ANALYSIS LIST

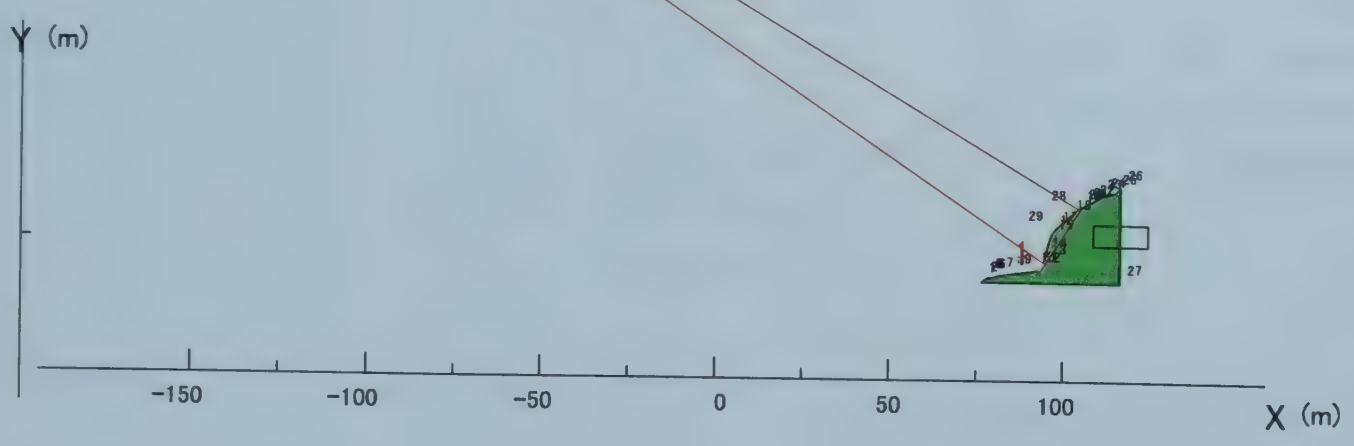
Ordinary condition					
Cohesion (kN/m²)	Angle (Degree)	tan(PHI)	Angle (Degree)	tan(PHI)	Cohesion (kN/m²)
0.0	46.31	1.05	0.00	0.00	28.40
1.0	45.29	1.01	1.00	0.02	27.93
2.0	44.22	0.97	2.00	0.03	27.46
3.0	43.12	0.94	3.00	0.05	26.98
4.0	41.97	0.90	4.00	0.07	26.51
5.0	40.78	0.86	5.00	0.09	26.03
6.0	39.55	0.83	6.00	0.11	25.55
7.0	38.27	0.79	7.00	0.12	25.07
8.0	36.95	0.75	8.00	0.14	24.59
9.0	35.57	0.72	9.00	0.16	24.11
10.0	34.15	0.68	10.00	0.18	23.62
11.0	32.68	0.64	11.00	0.19	23.13
12.0	31.16	0.60	12.00	0.21	22.64
13.0	29.59	0.57	13.00	0.23	22.14
14.0	27.97	0.53	14.00	0.25	21.64
15.0	26.29	0.49	15.00	0.27	21.13
16.0	24.57	0.46	16.00	0.29	20.62
17.0	22.80	0.42	17.00	0.31	20.11
18.0	20.98	0.38	18.00	0.32	19.59
19.0	19.12	0.35	19.00	0.34	19.06
20.0	17.21	0.31	20.00	0.36	18.53
21.0	15.26	0.27	21.00	0.38	17.99
22.0	13.28	0.24	22.00	0.40	17.44
23.0	11.27	0.20	23.00	0.42	16.89
24.0	9.22	0.16	24.00	0.45	16.33
25.0	7.15	0.13	25.00	0.47	15.75
26.0	5.06	0.09	26.00	0.49	15.17
27.0	2.96	0.05	27.00	0.51	14.58
28.0	0.85	0.01	28.00	0.53	13.98
29.0	0.00	0.00	29.00	0.55	13.37
30.0	0.00	0.00	30.00	0.58	12.74
31.0	0.00	0.00	31.00	0.60	12.10
32.0	0.00	0.00	32.00	0.62	11.45
33.0	0.00	0.00	33.00	0.65	10.79
34.0	0.00	0.00	34.00	0.67	10.10
35.0	0.00	0.00	35.00	0.70	9.41
36.0	0.00	0.00	36.00	0.73	8.69
37.0	0.00	0.00	37.00	0.75	7.96
38.0	0.00	0.00	38.00	0.78	7.21
39.0	0.00	0.00	39.00	0.81	6.43
40.0	0.00	0.00	40.00	0.84	5.64
41.0	0.00	0.00	41.00	0.87	4.82
42.0	0.00	0.00	42.00	0.90	3.98
43.0	0.00	0.00	43.00	0.93	3.11
44.0	0.00	0.00	44.00	0.97	2.21
45.0	0.00	0.00	45.00	1.00	1.27
46.0	0.00	0.00	46.00	1.04	0.31
47.0	0.00	0.00	47.00	1.07	0.00
48.0	0.00	0.00	48.00	1.11	0.00
49.0	0.00	0.00	49.00	1.15	0.00
50.0	0.00	0.00	50.00	1.19	0.00

Back Analysis Result

Loosen Slope B (Zone 6)

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....
Analysis type ..... Inverse analysis
Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{\sum W*\sin(a)}$$

Fs : Safety factor
C : Cohesion (kN/m²)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m²)
b : Width of a slice (m)
PHI : Friction angle (degree)

Width of a slice 10.0
Load condition ..... 0 (= 0 Affects both resisting and sliding forces)
(= 1 Only sliding force)

3. Inverse Analysis

INVERSE ANALYSIS LIST

Ordinary condition					
Cohesion (kN/m²)	Angle (Degree)	tan(PHI)	Angle (Degree)	tan(PHI)	Cohesion (kN/m²)
0.0	57.42	1.56	0.00	0.00	19.84
1.0	56.06	1.49	1.00	0.02	19.62
2.0	54.60	1.41	2.00	0.03	19.40
3.0	53.02	1.33	3.00	0.05	19.18
4.0	51.32	1.25	4.00	0.07	18.95
5.0	49.49	1.17	5.00	0.09	18.73
6.0	47.50	1.09	6.00	0.11	18.51
7.0	45.36	1.01	7.00	0.12	18.28
8.0	43.04	0.93	8.00	0.14	18.06
9.0	40.53	0.85	9.00	0.16	17.83
10.0	37.81	0.78	10.00	0.18	17.60
11.0	34.88	0.70	11.00	0.19	17.38
12.0	31.73	0.62	12.00	0.21	17.14
13.0	28.34	0.54	13.00	0.23	16.91
14.0	24.73	0.46	14.00	0.25	16.68
15.0	20.89	0.38	15.00	0.27	16.44
16.0	16.85	0.30	16.00	0.29	16.20
17.0	12.62	0.22	17.00	0.31	15.96
18.0	8.26	0.15	18.00	0.32	15.72
19.0	3.79	0.07	19.00	0.34	15.47
20.0	0.00	0.00	20.00	0.36	15.22
21.0	0.00	0.00	21.00	0.38	14.97
22.0	0.00	0.00	22.00	0.40	14.72
23.0	0.00	0.00	23.00	0.42	14.46
24.0	0.00	0.00	24.00	0.45	14.19
25.0	0.00	0.00	25.00	0.47	13.93
26.0	0.00	0.00	26.00	0.49	13.66
27.0	0.00	0.00	27.00	0.51	13.38
28.0	0.00	0.00	28.00	0.53	13.10
29.0	0.00	0.00	29.00	0.55	12.81
30.0	0.00	0.00	30.00	0.58	12.52
31.0	0.00	0.00	31.00	0.60	12.22
32.0	0.00	0.00	32.00	0.62	11.92
33.0	0.00	0.00	33.00	0.65	11.61
34.0	0.00	0.00	34.00	0.67	11.29
35.0	0.00	0.00	35.00	0.70	10.96
36.0	0.00	0.00	36.00	0.73	10.63
37.0	0.00	0.00	37.00	0.75	10.28
38.0	0.00	0.00	38.00	0.78	9.93
39.0	0.00	0.00	39.00	0.81	9.57
40.0	0.00	0.00	40.00	0.84	9.20
41.0	0.00	0.00	41.00	0.87	8.82
42.0	0.00	0.00	42.00	0.90	8.42
43.0	0.00	0.00	43.00	0.93	8.02
44.0	0.00	0.00	44.00	0.97	7.59
45.0	0.00	0.00	45.00	1.00	7.16
46.0	0.00	0.00	46.00	1.04	6.71
47.0	0.00	0.00	47.00	1.07	6.24
48.0	0.00	0.00	48.00	1.11	5.76
49.0	0.00	0.00	49.00	1.15	5.25
50.0	0.00	0.00	50.00	1.19	4.73

Appendix 1.2

Calculation of Rock Bolt

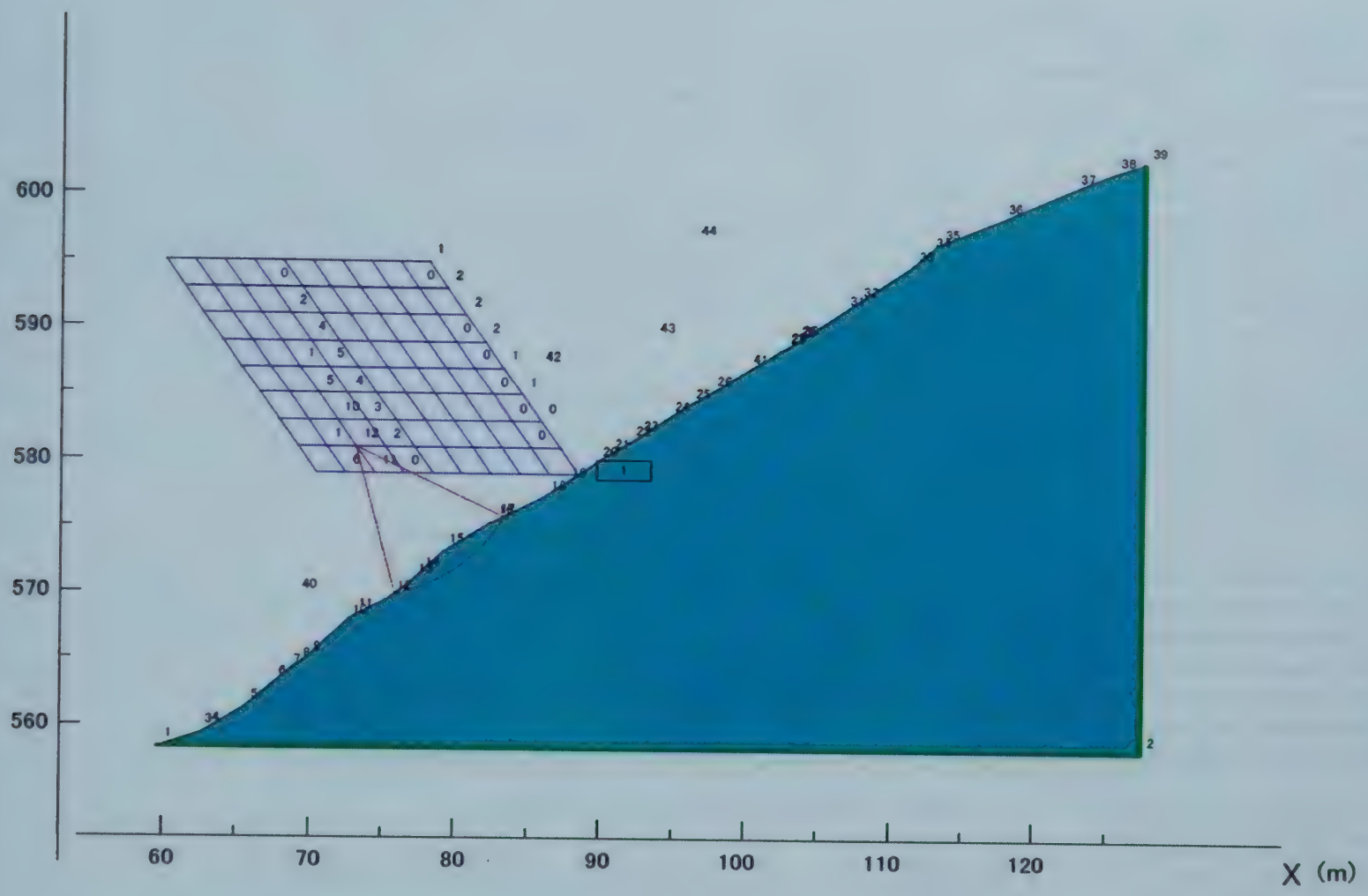
Calculation Result of Rock Bolt

Slide 1 (Zone 1, Zone 2, and Zone 4-1)

Calculation Result of Rock Bolt

Slide 1-Slip Surface 1 (Zone 1)

Surface with Maximum Prevention Force (Slip Surface 01)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-1 Slipsurface01(*maxPf)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	38	°	Cohesion	0.1 kN/m2
Resistance of landslide	ΣS	43.80	(kN/m)		
Sliding force	ΣT	50.20	(kN/m)	Safety factor	0.872
Necessary deterrent force	PR	16.4	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm) consideration of the corrosion allowance (nominal diameter−1.0mm)
		24.4	(mm)	consideration of the corrosion allowance (1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	
Horizontal spacing of reinforcement material	SH	4.0	(m)	
Vertical spacing of reinforcement material	SV	4.0	(m)	
Number of steps of reinforcement material	n	3	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa= (τp*π*D) /Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.501** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	43.8	
Sliding force	ΣT	(kN/m)	50.2	
Resistance power with reinforcemer	Sr	(kN/m)	31.55	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	2.32	1.08	3.40			90.75	436.56	61.30	93.52	61.30	42.91	10.73	0.00	8.38
(2)	2.03	1.37	3.40			96.00	382.27	77.58	93.52	77.58	54.31	13.58	0.00	10.55
(3)	1.46	1.95	3.40			99.37	274.26	109.99	93.52	93.52	65.46	16.37	0.00	12.62
(4)														
(5)														
(6)														
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													0.00	31.55

Calculating formula

- T1pa : Allowable tensile resistance force of reinforcement mataerial subjected to the moving mass — T1pa=(1/1-μ)*L1*ta
- T2pa : Allowable tensile resistance force of reinforcement mataerial subjected to the stable ground — T2pa=L2*ta
- Tsa : Allowable tensile stress of reinforcement mataerial — Tsa=σsa*As
- Tpa : Allowable tensile bearing strength of reinforcement mataerial — Tpa=min (T1pa, T2pa, Tsa)
- Td : Design tenaile force (1piece per unit) — Td=λ*Tpa
- Tm : Design tenaile force (1m width per unit) — Tm=Td/SH
- Sh : Detention force — Sh=Tm*cosβ
- SS : Striction force — Ss=Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 16.4 (kN/No.)

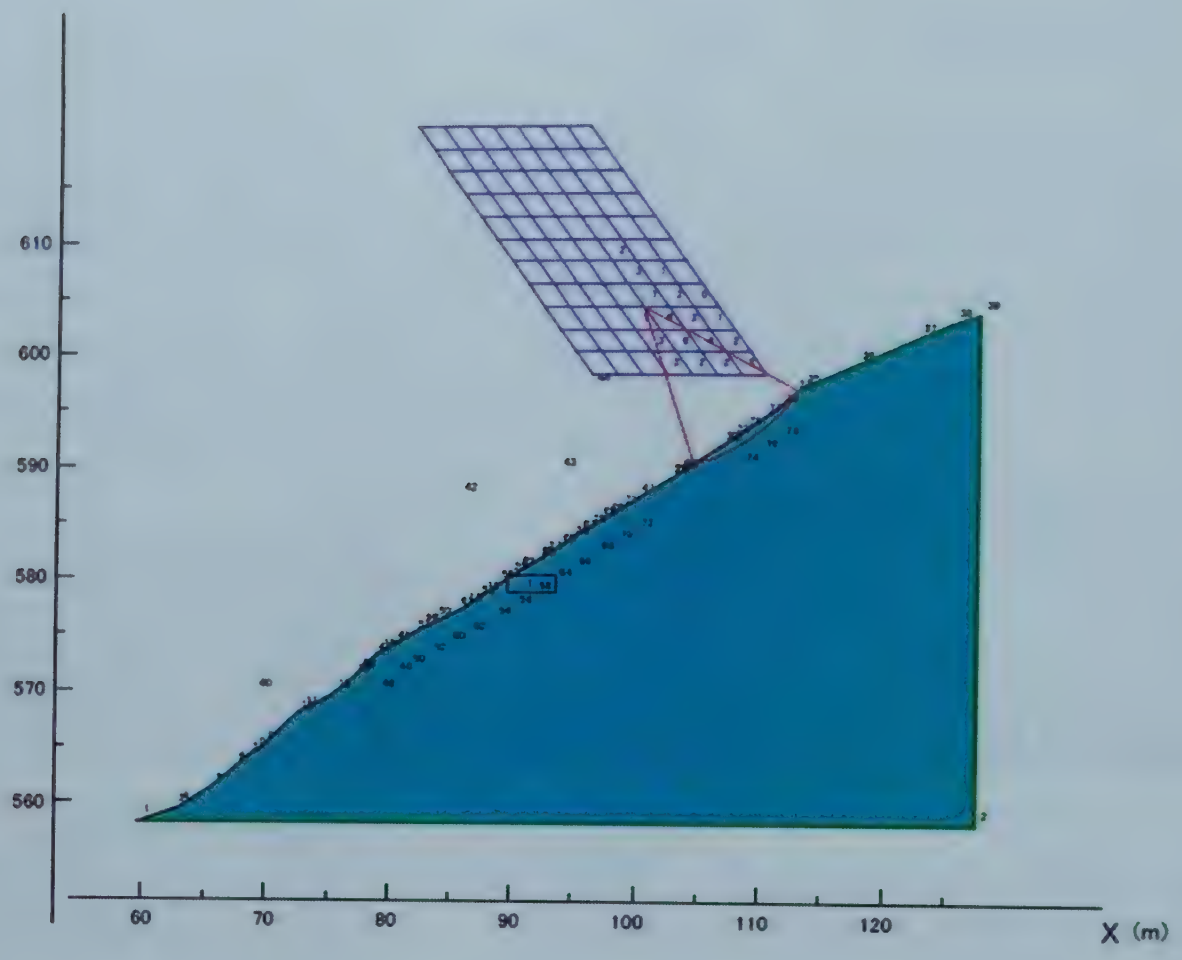
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.51 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.23 (m)

Calculation Result of Rock Bolt

Slide 1-Slip Surface 2 (Zone 1)

Surface with Maximum Prevention Force (Slip Surface 02)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-1 Slipsurface02(*maxPf)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	38	°	Cohesion	0.1 kN/m2
Resistance of landslide	ΣS	41.40	(kN/m)		
Sliding force	ΣT	41.40	(kN/m)	Safety factor	1.000
Necessary deterrent force	PR	8.3	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	4.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	3	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa=(τp*π*D)/Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.731** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	41.4	
Sliding force	ΣT	(kN/m)	41.4	
Resistance power with reinforcemer	Sr	(kN/m)	30.27	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.43	1.27	2.70			78.90	269.55	71.82	93.52	71.82	50.27	12.57	2.42	9.64
(2)	1.61	1.09	2.70			89.82	304.04	61.47	93.52	61.47	43.03	10.76	0.03	8.40
(3)	1.41	1.29	2.70			100.73	266.16	72.83	93.52	72.83	50.98	12.75	0.00	9.78
(4)														
(5)														
(6)														
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													2.45	27.82

Calculating formula

- T1pa : Allowable tensile resistance force of reinforcement mataerial subjected to the moving mass — T1pa=(1/1-μ)*L1*ta
- T2pa : Allowable tensile resistance force of reinforcement mataerial subjected to the stable ground — T2pa=L2*ta
- Tsa : Allowable tensile stress of reinforcement mataerial — Tsa=σsa*As
- Tpa : Allowable tensile bearing strength of reinforcement mataerial — Tpa=min (T1pa, T2pa, Tsa)
- Td : Design tensile force (1piece per unit) — Td=λ*Tpa
- Tm : Design tensile force (1m width per unit) — Tm=Td/SH
- Sh : Detention force — Sh=Tm*cosβ
- Ss : Striction force — Ss=Tm*sinβ*tanφ

Att) As stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 12.7 (kN/No.)

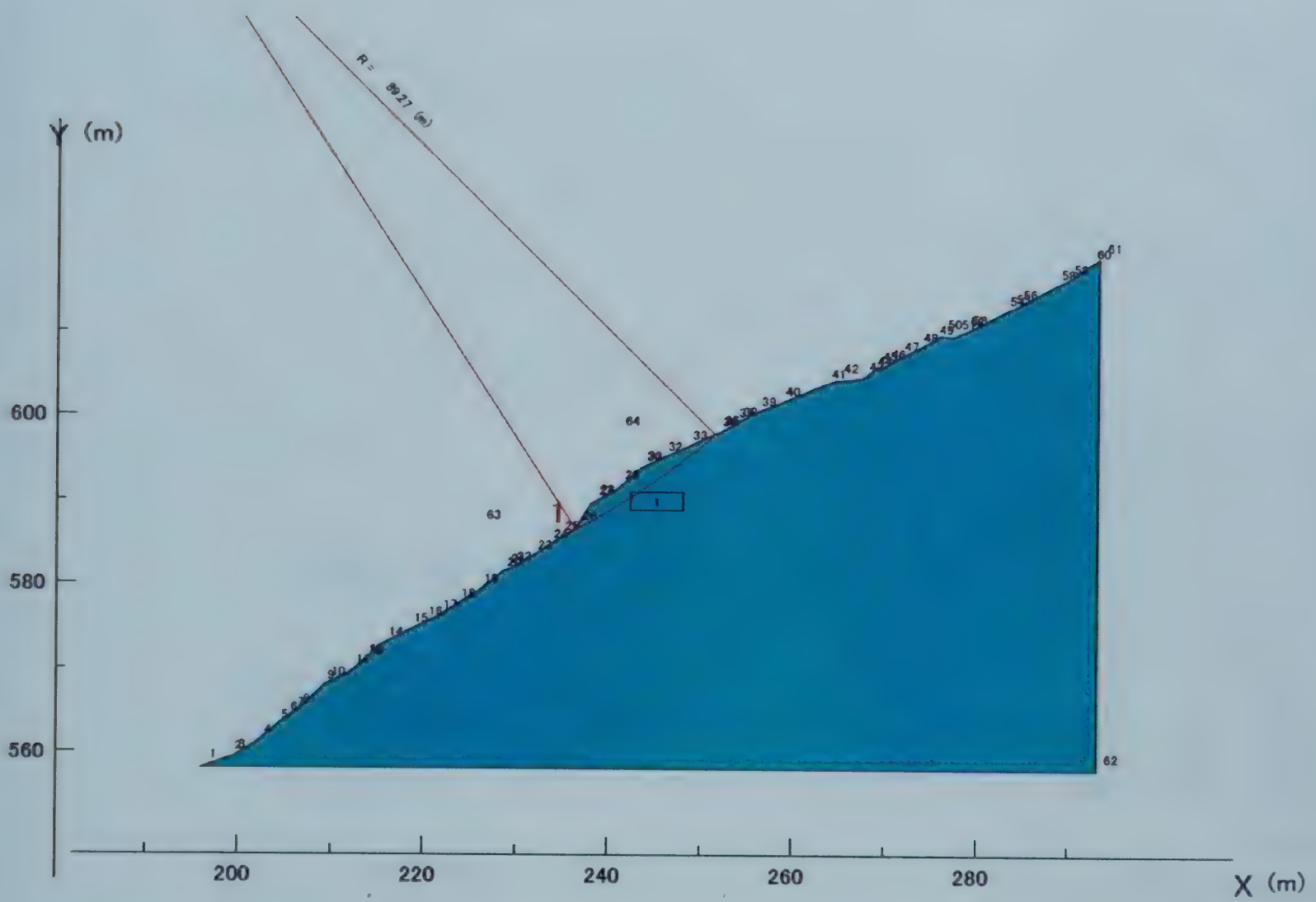
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.21 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.10 (m)

Calculation Result of Rock Bolt

Slide 1-Slip Surface 3 (Zone 4-1)

Surface with Maximum Prevention Force (Slip Surface 03)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-1 Slipsurface03(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	37	°	Cohesion	0 kN/m2
Resistance of landslide	ΣS	268.20	(kN/m)		
Sliding force	ΣT	260.10	(kN/m)	Safety factor	1.031
Necessary deterrent force	PR	43.9	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	4.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	4	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa= (τp*π*D) /Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.209** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	268.2	
Sliding force	ΣT	(kN/m)	260.1	
Resistance power with reinforcemer	Sr	(kN/m)	46.16	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.67	1.63	3.30			90.53	314.41	92.29	93.52	92.29	64.60	16.15	0.00	12.17
(2)	1.62	1.68	3.30			91.82	306.12	94.78	93.52	93.52	65.46	16.37	0.00	12.33
(3)	1.86	1.44	3.30			82.34	350.98	81.32	93.52	81.32	56.92	14.23	1.90	10.63
(4)	2.06	1.24	3.30			99.21	388.30	70.12	93.52	70.12	49.08	12.27	0.00	9.13
(5)														
(6)														
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													1.90	44.26

Calculating formula

- T1pa : Allowable tensile resistance force of reinforcement mataerial subjected to the moving mass — T1pa=(1/1-μ)*L1*ta
- T2pa : Allowable tensile resistance force of reinforcement mataerial subjected to the stable ground — T2pa=L2*ta
- Tsa : Allowable tensile stress of reinforcement mataerial — Tsa=σsa*As
- Tpa : Allowable tensile bearing strength of reinforcement mataerial — Tpa=min (T1pa, T2pa, Tsa)
- Td : Design tensile force (1piece per unit) — Td=λ*Tpa
- Tm : Design tensile force (1m width per unit) — Tm=Td/SH
- Sh : Detention force — Sh=Tm*cosβ
- SS : Striction force — Ss=Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 16.4 (kN/No.)

Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 1.05 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.47 (m)

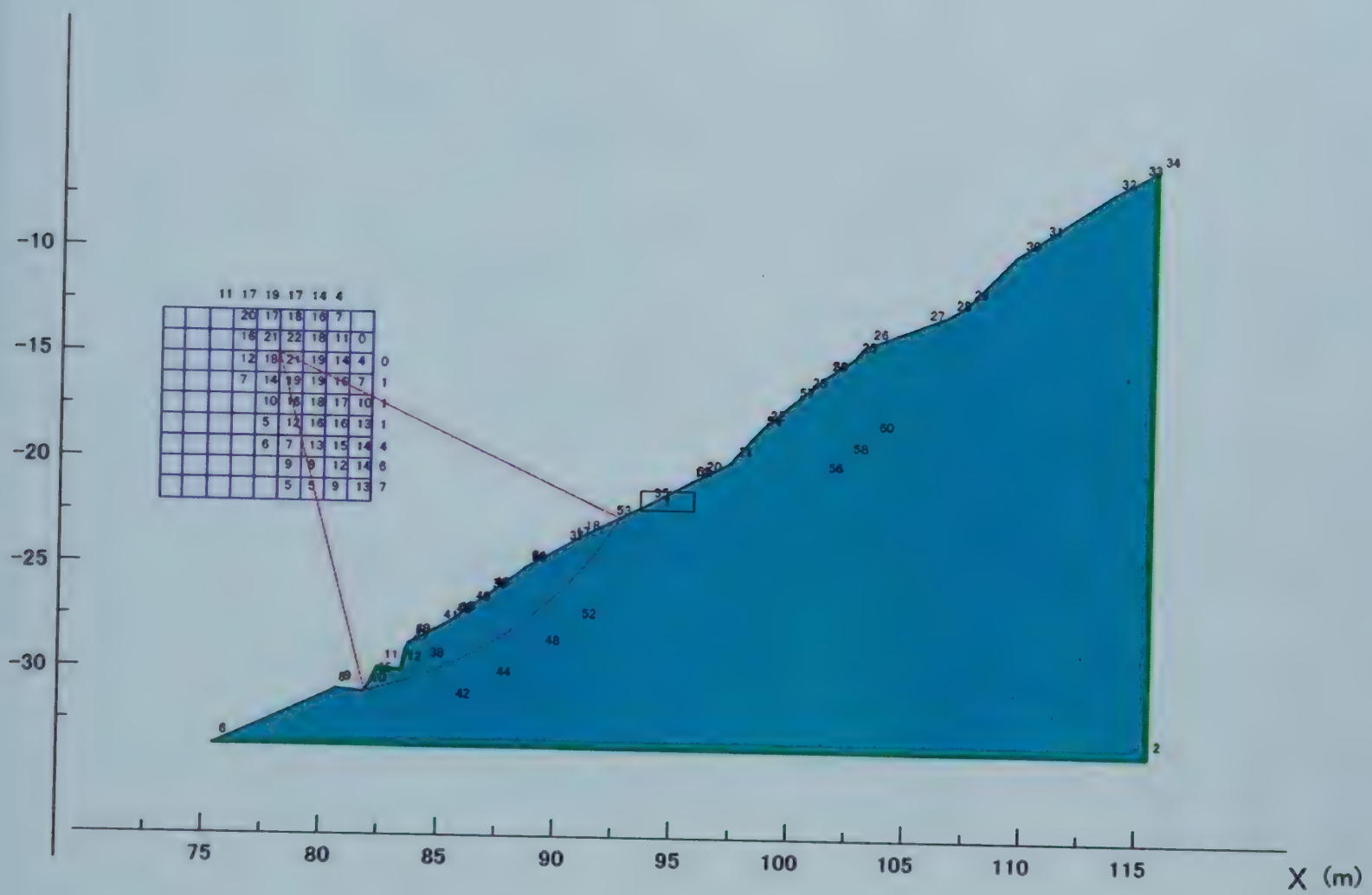
Calculation Result of Rock Bolt

Slide 2 (Zone 3 and Zone 4-2)

Calculation Result of Rock Bolt

Slide 2-Slip Surface 1 (Zone 3)

Surface with Maximum Prevention Force (Slip Surface 01)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-2 Slipsurface01(*maxPf)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	36	°	Cohesion	0.92 kN/m2
Resistance of landslide	ΣS	593.20	(kN/m)		
Sliding force	ΣT	570.50	(kN/m)	Safety factor	1.039
Necessary deterrent force	PR	91.4	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm) consideration of the corrosion allowance (nominal diameter−1.0mm)
		24.4	(mm)	consideration of the corrosion allowance (1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	
Horizontal spacing of reinforcement material	SH	2.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	6	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa=(τp*π*D)/Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.237** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	593.2	
Sliding force	ΣT	(kN/m)	570.5	
Resistance power with reinforcemer	Sr	(kN/m)	112.35	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.36	1.64	3.00			88.00	256.92	92.57	93.52	92.57	64.80	32.40	1.13	23.53
(2)	1.69	1.31	3.00			81.53	318.75	74.02	93.52	74.02	51.81	25.91	3.82	18.62
(3)	1.98	1.02	3.00			82.89	373.03	57.74	93.52	57.74	40.42	20.21	2.50	14.57
(4)	2.00	1.01	3.00			90.09	376.05	56.83	93.52	56.83	39.78	19.89	0.00	14.45
(5)	1.92	1.08	3.00			94.36	361.16	61.30	93.52	61.30	42.91	21.46	0.00	15.54
(6)	1.73	1.27	3.00			95.72	325.91	71.87	93.52	71.87	50.31	25.16	0.00	18.19
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													7.45	104.90

Calculating formula

T1pa :	Allowable tensile resistance force of reinforcement mataerial subjected to the moving mass	T1pa=(1/1-μ)*L1*ta	Td :	Design tensile force (1piece per unit)	Td=λ*Tpa
T2pa :	Allowable tensile resistance force of reinforcement mataerial subjected to the stable ground	T2pa=L2*ta	Tm :	Design tensile force (1m width per unit)	Tm=Td/SH
Tsa :	Allowable tensile stress of reinforcement mataerial	Tsa=σsa*As	Sh :	Detention force	Sh=Tm*cosβ
Tpa :	Allowable tensile bearing strength of reinforcement mataerial	Tpa=min (T1pa, T2pa, Tsa)	SS :	Striction force	Ss=Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 32.4 (kN/No.)

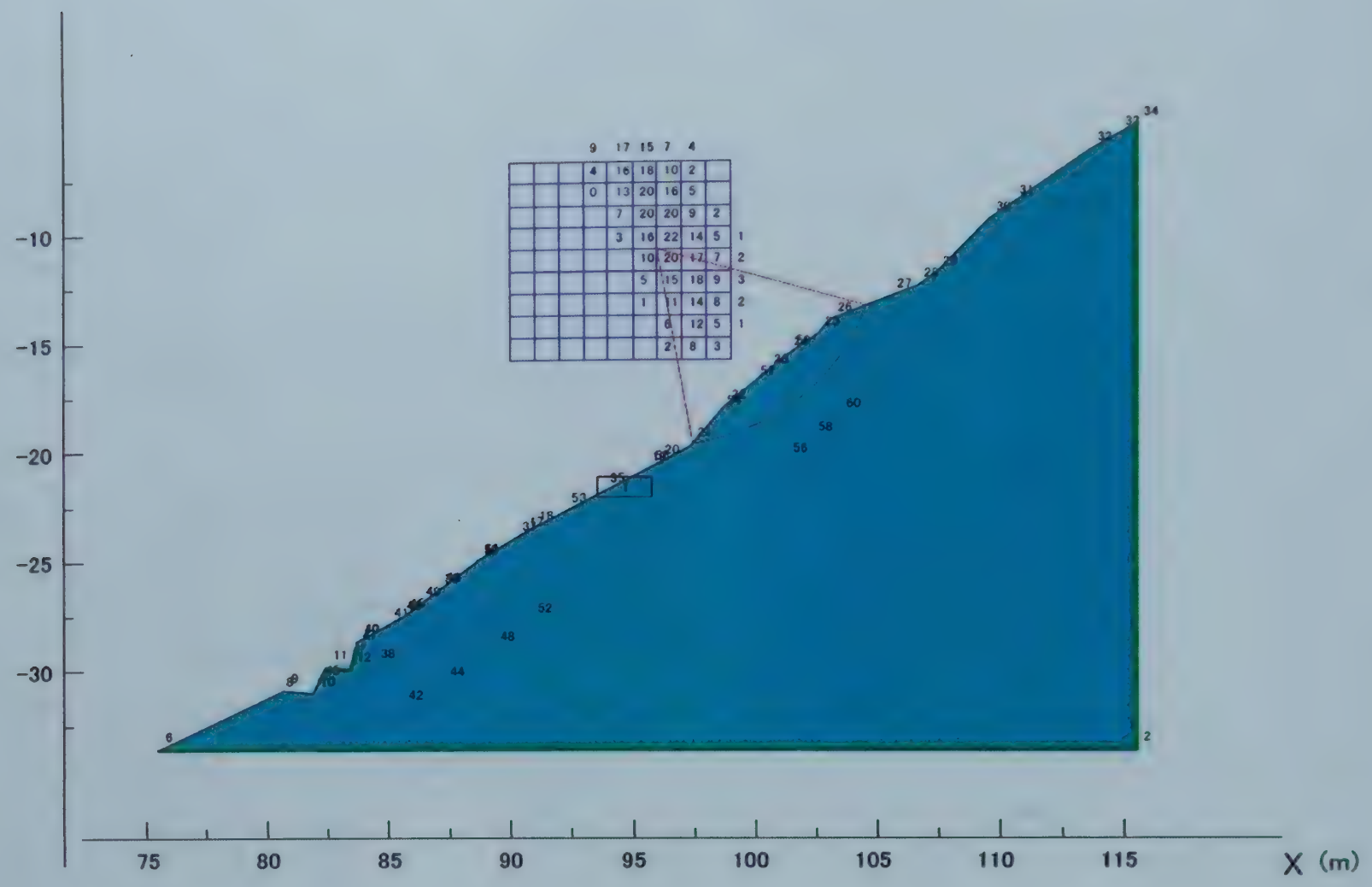
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.71 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.32 (m)

Calculation Result of Rock Bolt

Slide 2-Slip Surface 2 (Zone 4-2)

Surface with Maximum Prevention Force (Slip Surface 02)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-2 Slipsurface02(*maxPf)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor						
Plan safety factor of reinforcement slope	Fs	1.20				
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345		
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)		
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360	(kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0				
Reduction coefficient for tensile force	λ	0.7				
slope protection reduction coefficient	μ	0.7				

Condition of slope						
Internal friction angle of soil	φ	36	°	Cohesion	0.92	kN/m2
Resistance of landslide	ΣS	263.60	(kN/m)			
Sliding force	ΣT	274.40	(kN/m)	Safety factor	0.960	
Necessary deterrent force	PR	65.7	(kN/m)	PR=Fs*ΣT-ΣS		

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm) consideration of the corrosion allowance (nominal diameter−1.0mm)
		24.4	(mm)	
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	2.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	4	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa=(τp*π*D)/Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.272** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	263.6	
Sliding force	ΣT	(kN/m)	274.4	
Resistance power with reinforcemer	Sr	(kN/m)	85.53	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.61	1.40	3.00			79.21	302.54	78.89	93.52	78.89	55.22	27.61	5.17	19.71
(2)	1.70	1.30	3.00			88.38	319.88	73.68	93.52	73.68	51.58	25.79	0.73	18.73
(3)	1.73	1.27	3.00			96.56	326.47	71.70	93.52	71.70	50.19	25.10	0.00	18.11
(4)	1.64	1.37	3.00			82.28	308.19	77.19	93.52	77.19	54.03	27.02	3.63	19.45
(5)														
(6)														
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													9.53	76.00

Calculating formula

T1pa :	Allowable tensile resistance force of reinforcement material subjected to the moving mass	T1pa=(1/1-μ)*L1*ta	Td :	Design tensile force (1piece per unit)	Td=λ*Tpa
T2pa :	Allowable tensile resistance force of reinforcement material subjected to the stable ground	T2pa=L2*ta	Tm :	Design tensile force (1m width per unit)	Tm=Td/SH
Tsa :	Allowable tensile stress of reinforcement material	Tsa=σsa*As	Sh :	Detention force	Sh=Tm*cosβ
Tpa	Allowable tensile bearing strength of reinforcement material	Tpa=min (T1pa, T2pa, Tsa)	SS :	Striction force	Ss=Tm*sinβ*tanφ

All) As stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 27.6 (kN/No.)

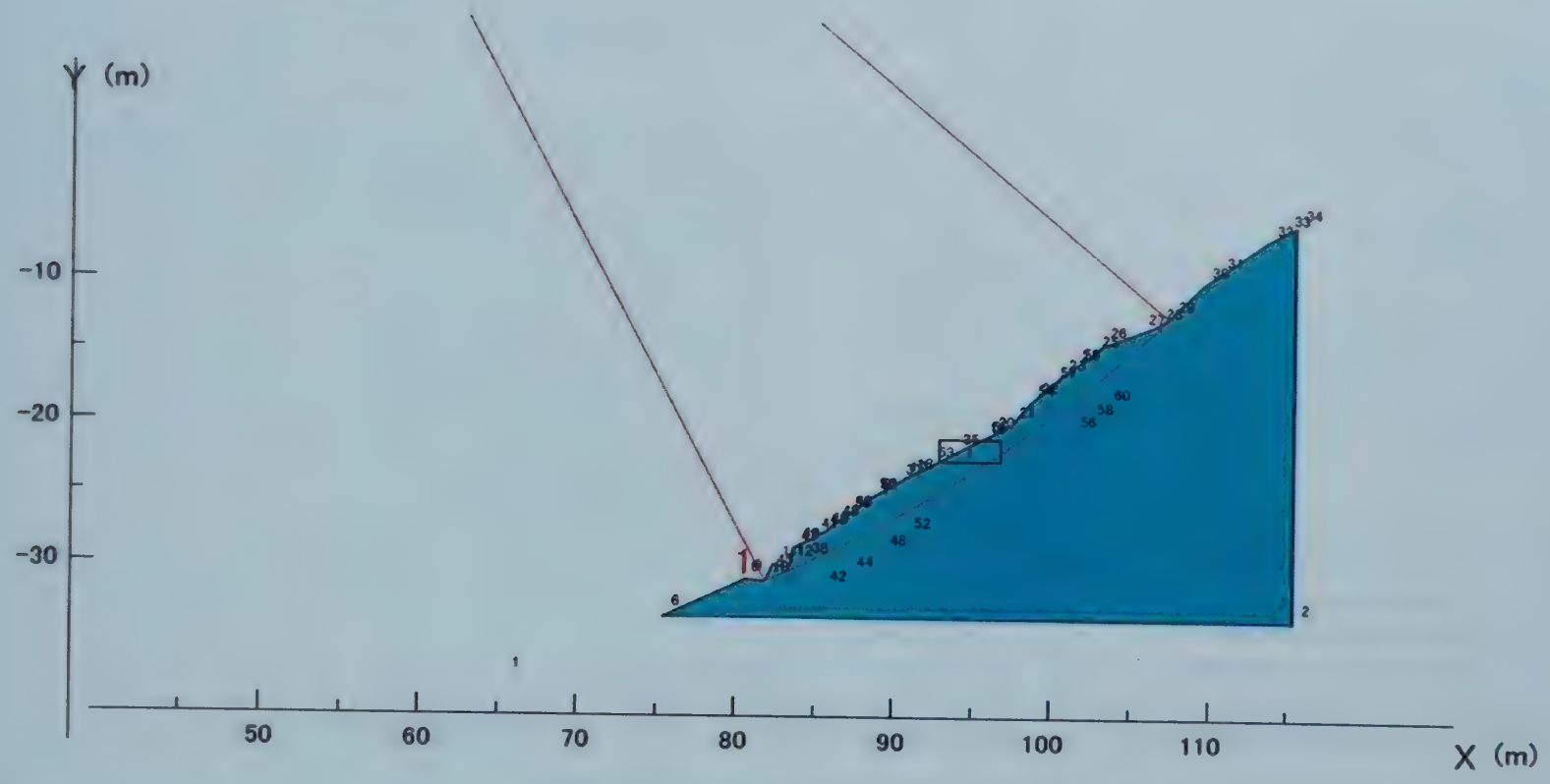
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.65 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.29 (m)

Calculation Result of Rock Bolt

Slide 2-Slip Surface 3 (Zone 3 and Zone 4-2)

Surface with Maximum Prevention Force (Slip Surface 03)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-2 Slipsurface03(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	36	°	Cohesion	0.92 kN/m2
Resistance of landslide	ΣS	505.70	(kN/m)		
Sliding force	ΣT	474.50	(kN/m)	Safety factor	1.065
Necessary deterrent force	PR	63.7	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	2.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	10	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa= (τp*π*D) /Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.483** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	505.7	
Sliding force	ΣT	(kN/m)	474.5	
Resistance power with reinforcemer	Sr	(kN/m)	197.88	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.36	1.64	3.00			88.00	256.92	92.57	93.52	92.57	64.80	32.40	1.13	23.53
(2)	1.69	1.31	3.00			81.53	318.75	74.02	93.52	74.02	51.81	25.91	3.82	18.62
(3)	1.98	1.02	3.00			82.89	373.03	57.74	93.52	57.74	40.42	20.21	2.50	14.57
(4)	2.00	1.01	3.00			90.09	376.05	56.83	93.52	56.83	39.78	19.89	0.00	14.45
(5)	1.92	1.08	3.00			94.36	361.16	61.30	93.52	61.30	42.91	21.46	0.00	15.54
(6)	1.73	1.27	3.00			95.72	325.91	71.87	93.52	71.87	50.31	25.16	0.00	18.19
(7)														
(8)														
(9)	1.61	1.40	3.00			79.21	302.54	78.89	93.52	78.89	55.22	27.61	5.17	19.71
(10)	1.70	1.30	3.00			88.38	319.88	73.68	93.52	73.68	51.58	25.79	0.73	18.73
(11)	1.73	1.27	3.00			96.56	326.47	71.70	93.52	71.70	50.19	25.10	0.00	18.11
(12)	1.64	1.37	3.00			82.28	308.19	77.19	93.52	77.19	54.03	27.02	3.63	19.45
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													16.98	180.90

Calculating formula

T1pa :	Allowable tensile resistance force of reinforcement mataenal subjected to the moving mass	T1pa=(1/1-μ)*L1*ta	Td :	Design tensile force (1piece per unit)	Td=λ*Tpa
T2pa :	Allowable tensile resistance force of reinforcement mataerial subjected to the stable ground	T2pa=L2*ta	Tm :	Design tensile force (1m width per unit)	Tm=Td/SH
Tsa :	Allowable tensile stress of reinforcement mataenal	Tsa=σsa*As	Sh :	Detention force	Sh=Tm*cosβ
Tpa :	Allowable tensile bearing strength of reinforcement mataerial	Tpa=min (T1pa, T2pa, Tsa)	SS :	Striction force	Ss=Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 32.4 (kN/No.)

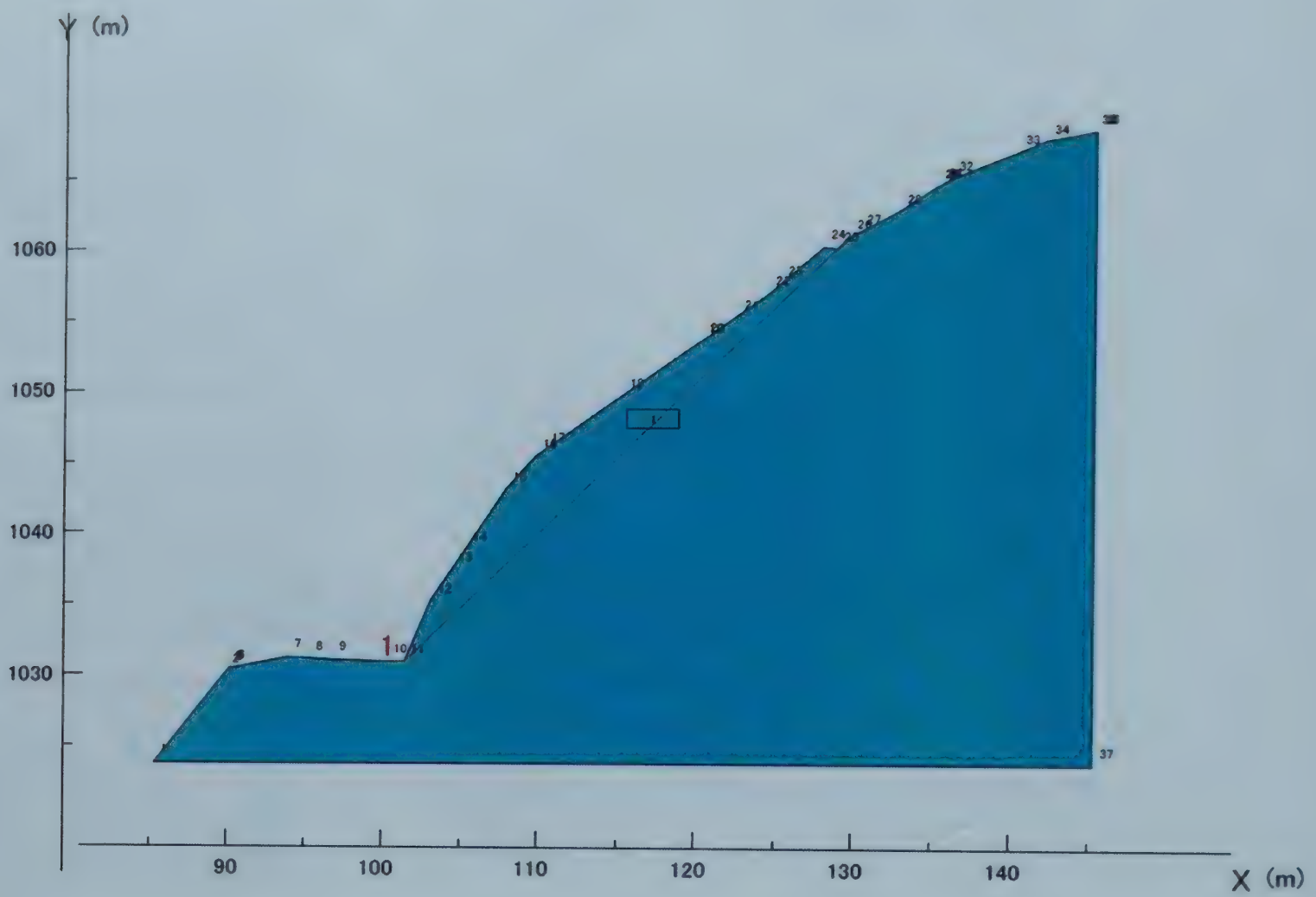
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.30 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.14 (m)

Calculation Result of Rock Bolt

Loosen Slope A (Zone 7)

Surface with Maximum Prevention Force



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Loosen slope A(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor						
Plan safety factor of reinforcement slope	Fs	1.20				
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345		
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)		
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360	(kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0				
Reduction coefficient for tensile force	λ	0.7				
slope protection reduction coefficient	μ	0.7				

Condition of slope						
Internal friction angle of soil	φ	38	°	Cohesion	7.21	kN/m2
Resistance of landslide	ΣS	1129.30	(kN/m)			
Sliding force	ΣT	1129.20	(kN/m)	Safety factor	1.000	
Necessary deterrent force	PR	225.7	(kN/m)	PR=Fs*ΣT-ΣS		

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	2.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	8	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa= (τp*π*D) /Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result		
Safety factor after reinforcement	Fs		**1.209**	> 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	1129.3		
Sliding force	ΣT	(kN/m)	1129.2		
Resistance power with reinforcemer	Sr	(kN/m)	235.67		(Sh+Ss)

Conditions of performance / Design of tensile force of the each reinforcement

Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	0.95	3.95	4.90			68.31	179.26	223.31	93.52	93.52	65.46	32.73	12.10	23.76
(2)	1.76	3.14	4.90			68.31	331.19	177.73	93.52	93.52	65.46	32.73	12.10	23.76
(3)	2.10	2.80	4.90			78.31	396.03	158.28	93.52	93.52	65.46	32.73	6.63	25.04
(4)	2.54	2.36	4.90			77.31	478.40	133.57	93.52	93.52	65.46	32.73	7.19	24.95
(5)	2.97	1.93	4.90			77.31	560.21	109.03	93.52	93.52	65.46	32.73	7.19	24.95
(6)	3.41	1.50	4.90			77.31	641.83	84.54	93.52	84.54	59.18	29.59	6.50	22.55
(7)	3.68	1.22	4.90			85.31	693.66	68.99	93.52	68.99	48.29	24.15	1.97	18.80
(8)	3.83	1.07	4.90			85.31	722.50	60.34	93.52	60.34	42.24	21.12	1.73	16.45
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15,														
(16)														
(17)														
(18)														
Sum													55.41	180.26

Calculating formula

- T1pa : Allowable tensile resistance force of reinforcement materail subjected to the moving mass — T1pa=(1/1-μ)*L1*ta
- T2pa : Allowable tensile resistance force of reinforcement materail subjected to the stable ground — T2pa=L2*ta
- Tsa : Allowable tensile stress of reinforcement materail — Tsa=σsa*As
- Tpa : Allowable tensile bearing strength of reinforcement materail — Tpa= min (T1pa, T2pa, Tsa)
- Td : Design tensile force (1piece per unit) — Td=λ*Tpa
- Tm : Design tensile force (1m width per unit) — Tm=Td/SH
- Sh : Detention force — Sh=Tm*cosβ
- Ss : Striction force — Ss=Tm*sinβ*tanφ

Att) As stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 32.7 (kN/No.)

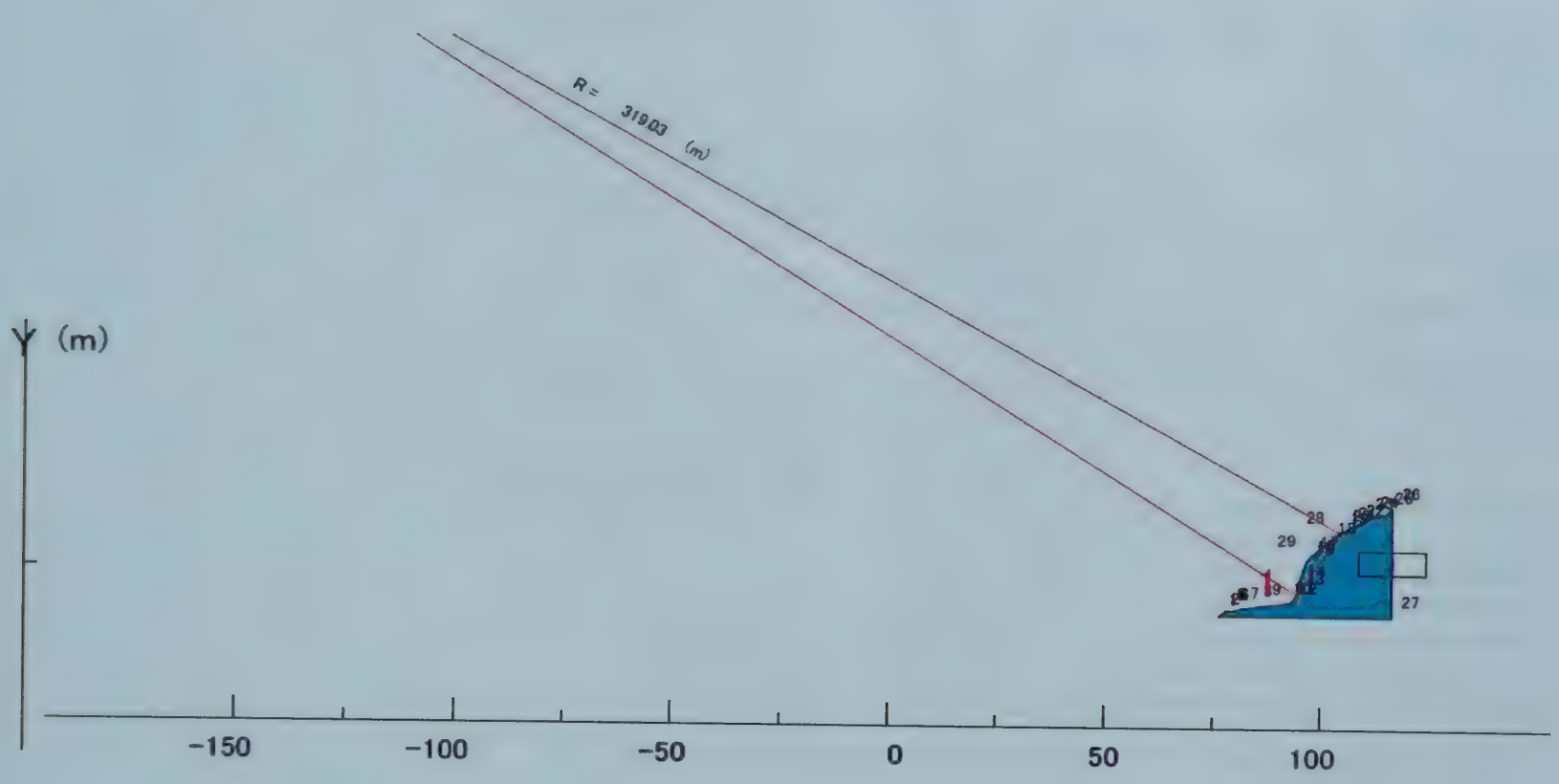
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.92 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.41 (m)

Calculation Result of Rock Bolt

Loosen Slope B (Zone 6)

Surface with Maximum Prevention Force



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Loosen slope B(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	360	(kN/m2)	τp1	360 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	38	°	Cohesion	9.93 kN/m2
Resistance of landslide	ΣS	420.50	(kN/m)		
Sliding force	ΣT	420.60	(kN/m)	Safety factor	0.999
Necessary deterrent force	PR	84.2	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	2.0	(m)	
Vertical spacing of reinforcement material	SV	2.0	(m)	
Number of steps of reinforcement material	n	5	(Step)	
Allowable bond stress of reinforcement material	ta	56.549	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	56.549	(kN/m)	tpa= (τp*π*D) /Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.334** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	420.5	
Sliding force	ΣT	(kN/m)	420.6	
Resistance power with reinforcemer	Sr	(kN/m)	140.52	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	1.04	2.56	3.60			64.50	195.28	144.99	93.52	93.52	65.46	32.73	14.09	23.08
(2)	1.44	2.17	3.60			74.78	270.49	122.43	93.52	93.52	65.46	32.73	8.59	24.67
(3)	1.97	1.63	3.60			75.15	372.09	91.95	93.52	91.95	64.37	32.19	8.25	24.31
(4)	2.50	1.10	3.60			75.52	470.48	62.43	93.52	62.43	43.70	21.85	5.46	16.53
(5)	2.59	1.01	3.60			93.72	488.77	56.94	93.52	56.94	39.86	19.93	0.00	15.54
(6)														
(7)														
(8)														
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													36.39	104.13

Calculating formula

T1pa :	Allowable tensile resistance force of reinforcement material subjected to the moving mass	T1pa=(1/1-μ)*L1*ta	Td :	Design tensile force (1piece per unit)	Td=λ*Tpa
T2pa :	Allowable tensile resistance force of reinforcement material subjected to the stable ground	T2pa=L2*ta	Tm :	Design tensile force (1m width per unit)	Tm=Td/SH
Tsa :	Allowable tensile stress of reinforcement material	Tsa=σsa*As	Sh :	Detention force	Sh=Tm*cosβ
Tpa :	Allowable tensile bearing strength of reinforcement material	Tpa=min (T1pa, T2pa, Tsa)	SS :	Striction force	Ss Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 32.7 (kN/No.)

Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.53 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.24 (m)

Appendix 1.3

Calculation of Crib Works

Calculation of Crib Works

Slide 1 (Zone 1, Zone 2, and Zone 4-1)

Calculation of Crib Works

Slide 1-Slip Surface 1 (Zone 1)

Design load	Td=	16.4	kN／No.	(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reintocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar		D13	×上下各	2 本
		126.7		

Calculation result

Uniformly-distributed load	w=	4.43	kN/m
Maximum bending moment	M=	2.22	kN・m
Maximum shear strength	S=	4.43	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	1.1	N/mm2	OK	6
◎Tensile stress of reinforcing bar	σs=	41.2	N/mm2	OK	180
◎Shear stress of mortar	τc=	0.07	N/mm2	OK	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.13	N/mm2	OK	1.4

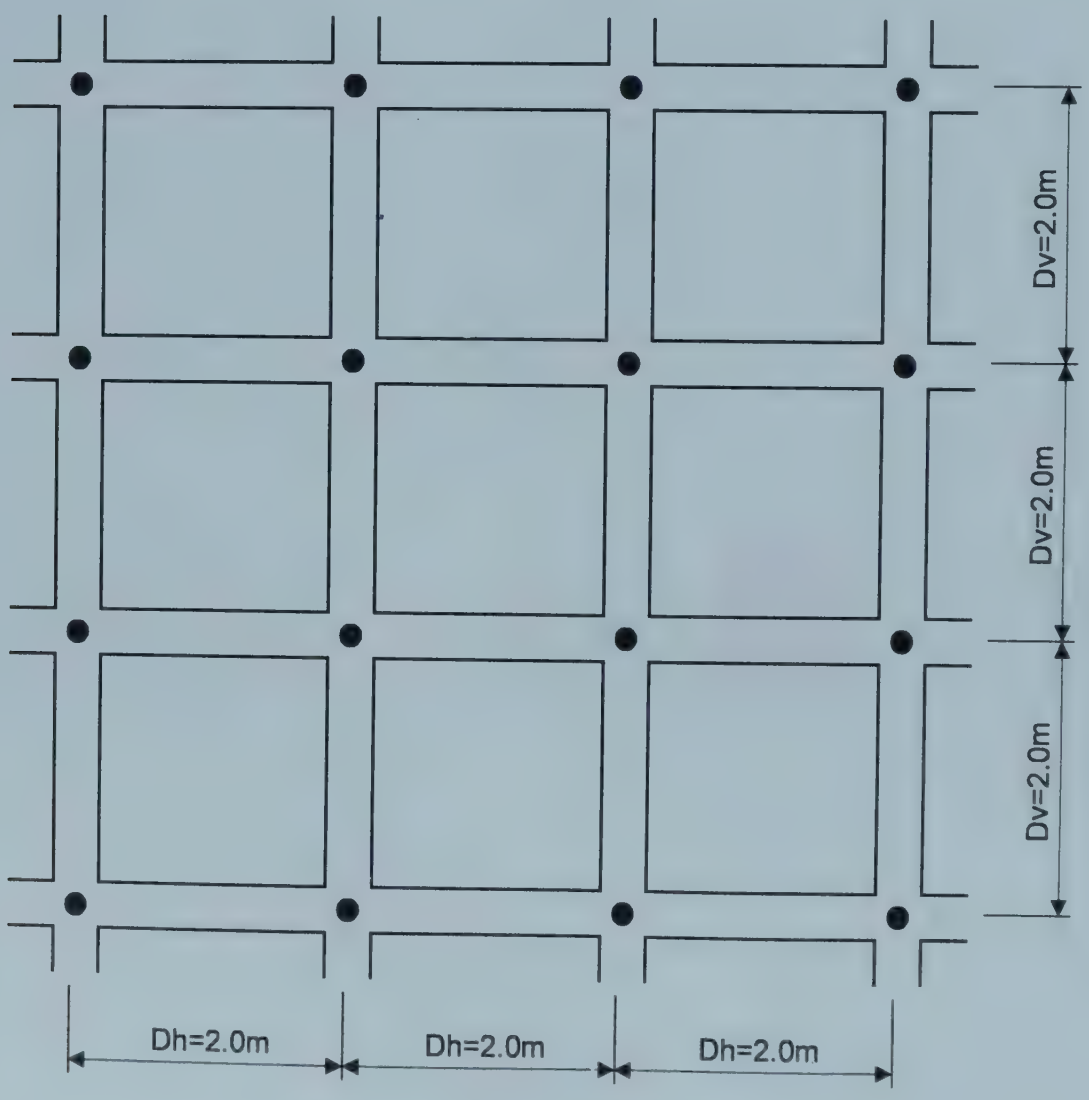
Veight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	16.4	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{16.4}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 4.43 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 4.43 \times 2.00^{2}$$

$$= 2.22 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 4.43 \times 2.00$$

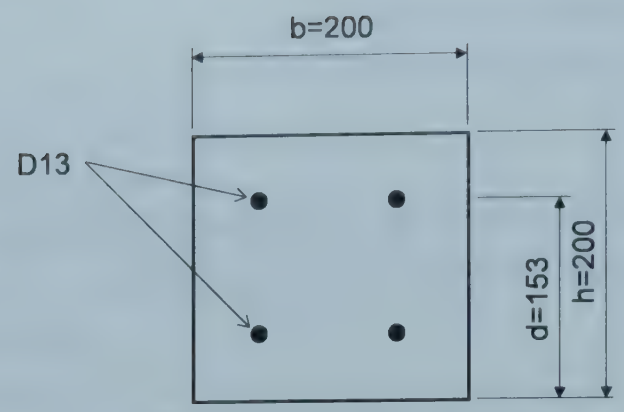
$$= 4.43 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13	×	Upper and lov 2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 1.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 41.2 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{4.43 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.07 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{4.43 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.26 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Slide 1-Slip Surface 2 (Zone 1)

Design load	Td=	12.7	kN／No. (Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reinfocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar: D13 ×上下各 2 本

126.7

Calculation result

Uniformly-distributed load	w=	3.43	kN/m
Maximum bending moment	M=	1.72	kN･m
Maximum shear strength	S=	3.43	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	0.8	N/mm2	OK	6
◎Tensile stress of reinforcing bar	σs=	31.9	N/mm2	OK	180
◎Shear stress of mortar	τc=	0.05	N/mm2	OK	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.10	N/mm2	OK	1.4

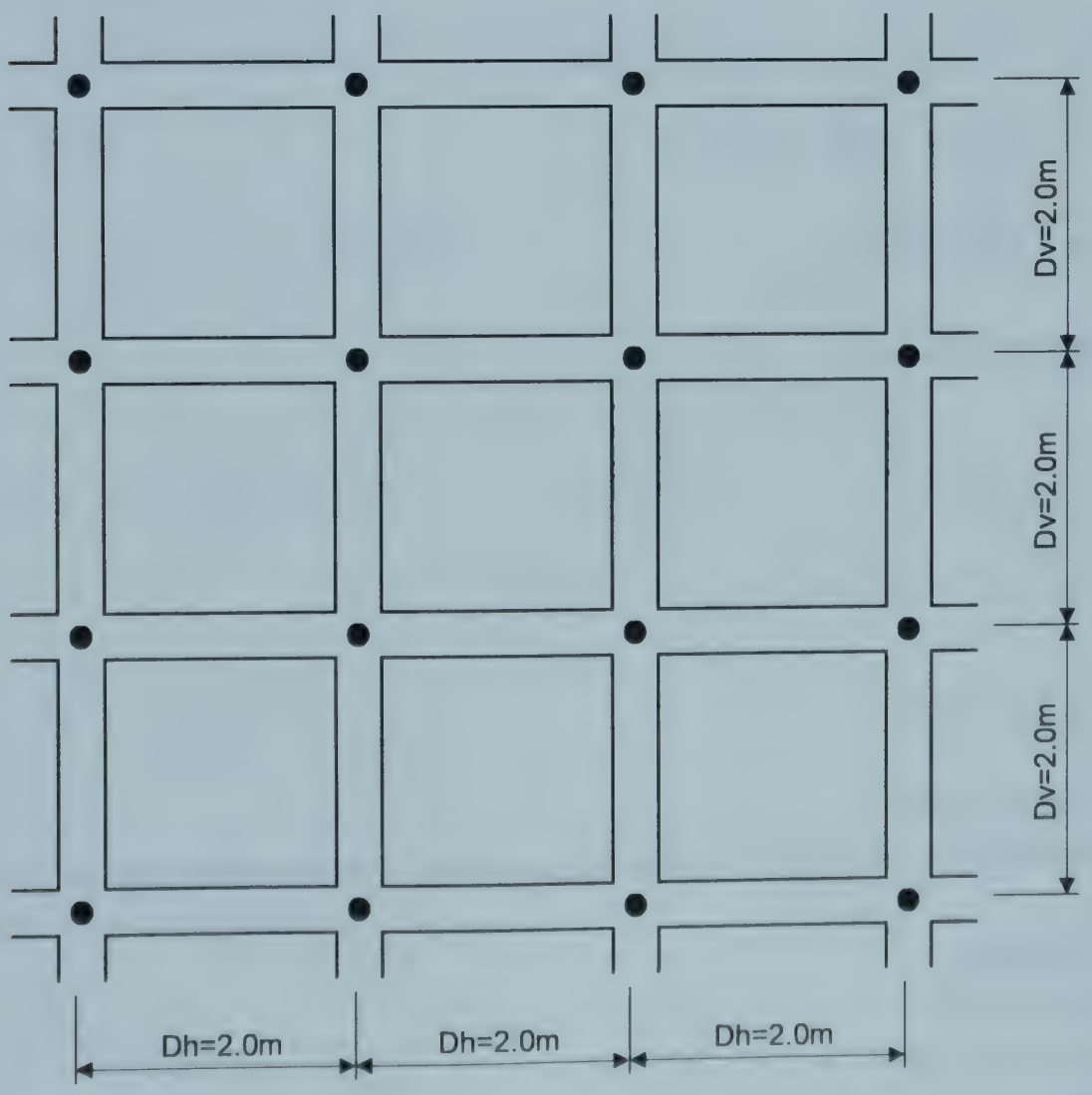
Weight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	12.7	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{12.7}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 3.43 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 3.43 \times 2.00^2$$

$$= 1.72 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 3.43 \times 2.00$$

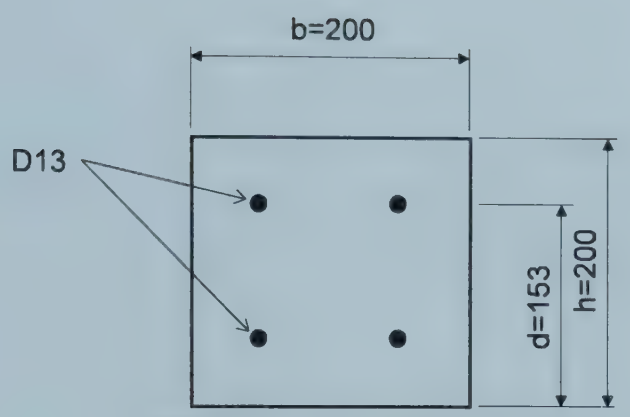
$$= 3.43 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13 ×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{1.72 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 0.8 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{1.72 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 31.9 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{3.43 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.05 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{3.43 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.20 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Slide 1-Slip Surface 3 (Zone 1)

Design load	Td=	16.4	kN／No.	(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reintocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar		D13	×上下各	2 本
		126.7		

Calculation result

Uniformly-distributed load	w=	4.43	kN/m
Maximum bending moment	M=	2.22	kN･m
Maximum shear strength	S=	4.43	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	1.1	N/mm2	O K	6
◎Tensile stress of reinforcing bar	σs=	41.2	N/mm2	O K	180
◎Shear stress of mortar	τc=	0.07	N/mm2	O K	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.13	N/mm2	O K	1.4

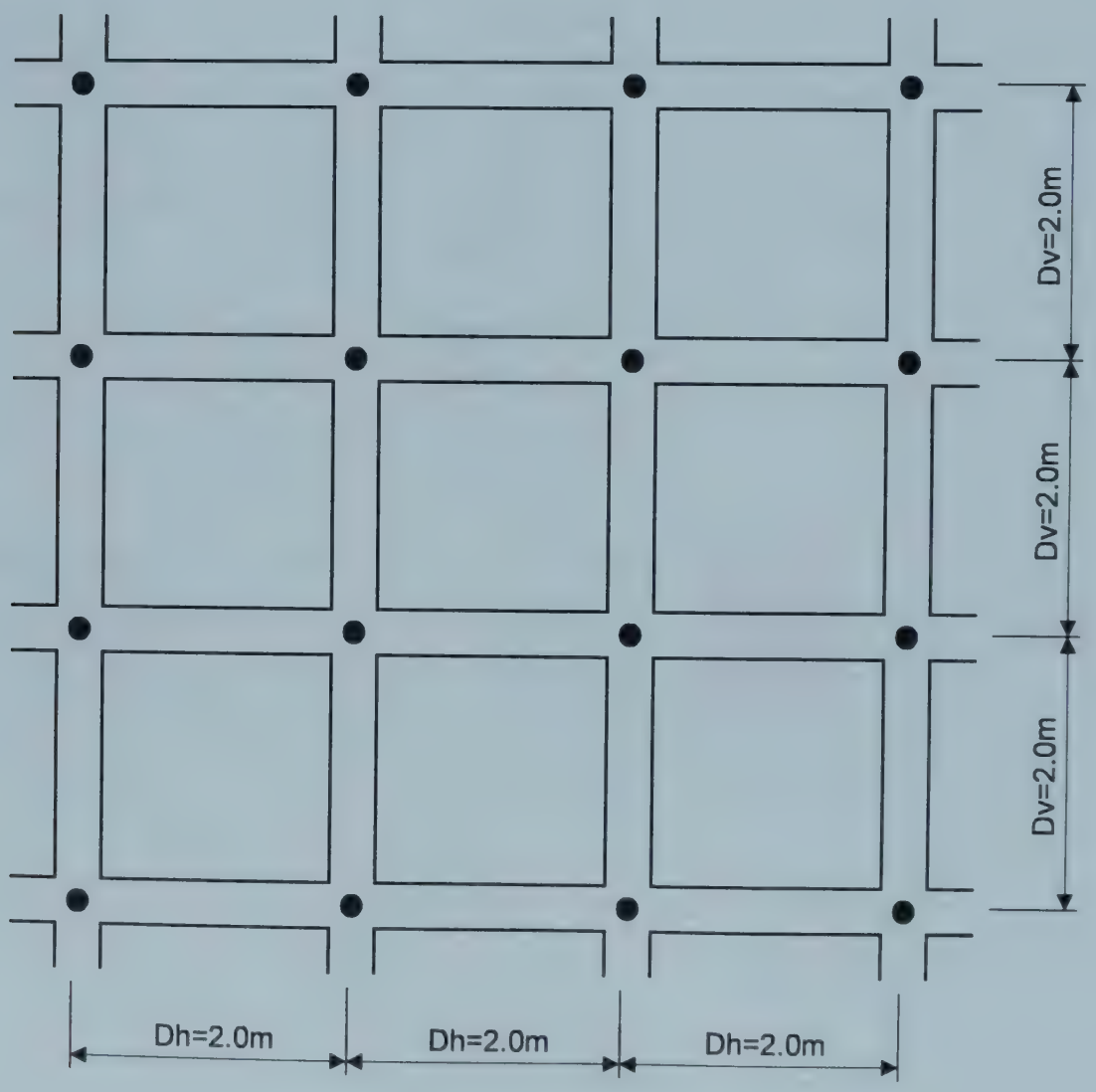
Veight, sectional area, and peripheral length of strange appearance steel b:

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	16.4	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{16.4}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 4.43 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 4.43 \times 2.00^{2}$$

$$= 2.22 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 4.43 \times 2.00$$

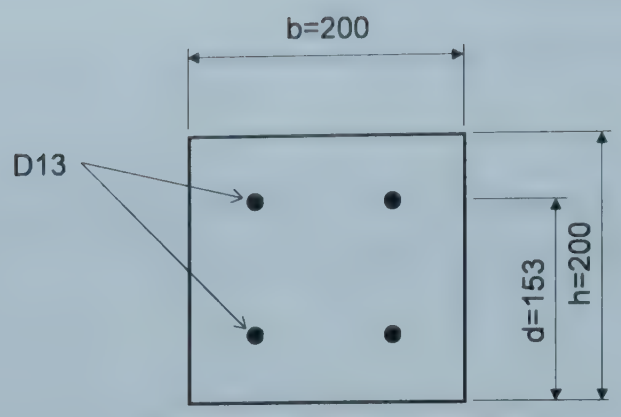
$$= 4.43 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300	mm
Head space of the frame	d	=	235	mm
Reinforcing bar	D13	×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 1.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 41.2 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{4.43 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.07 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar = # mm × 2 Pieces

$$= \frac{4.43 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.26 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Slide 2 (Zone 3 and Zone 4-2)

Calculation of Crib Works

Slide 2-Slip Surface 1 (Zone 3)

Design load	Td=	32.4	kN／No. (Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reinfocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar: D13 ×上下各 2 本

126.7

Calculation result

Uniformly-distributed load	w=	8.76	kN/m
Maximum bending moment	M=	4.38	kN･m
Maximum shear strength	S=	8.76	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	2.1	N/mm2	O K	6
◎Tensile stress of reinforcing bar	σs=	81.2	N/mm2	O K	180
◎Shear stress of mortar	τc=	0.14	N/mm2	O K	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.26	N/mm2	O K	1.4

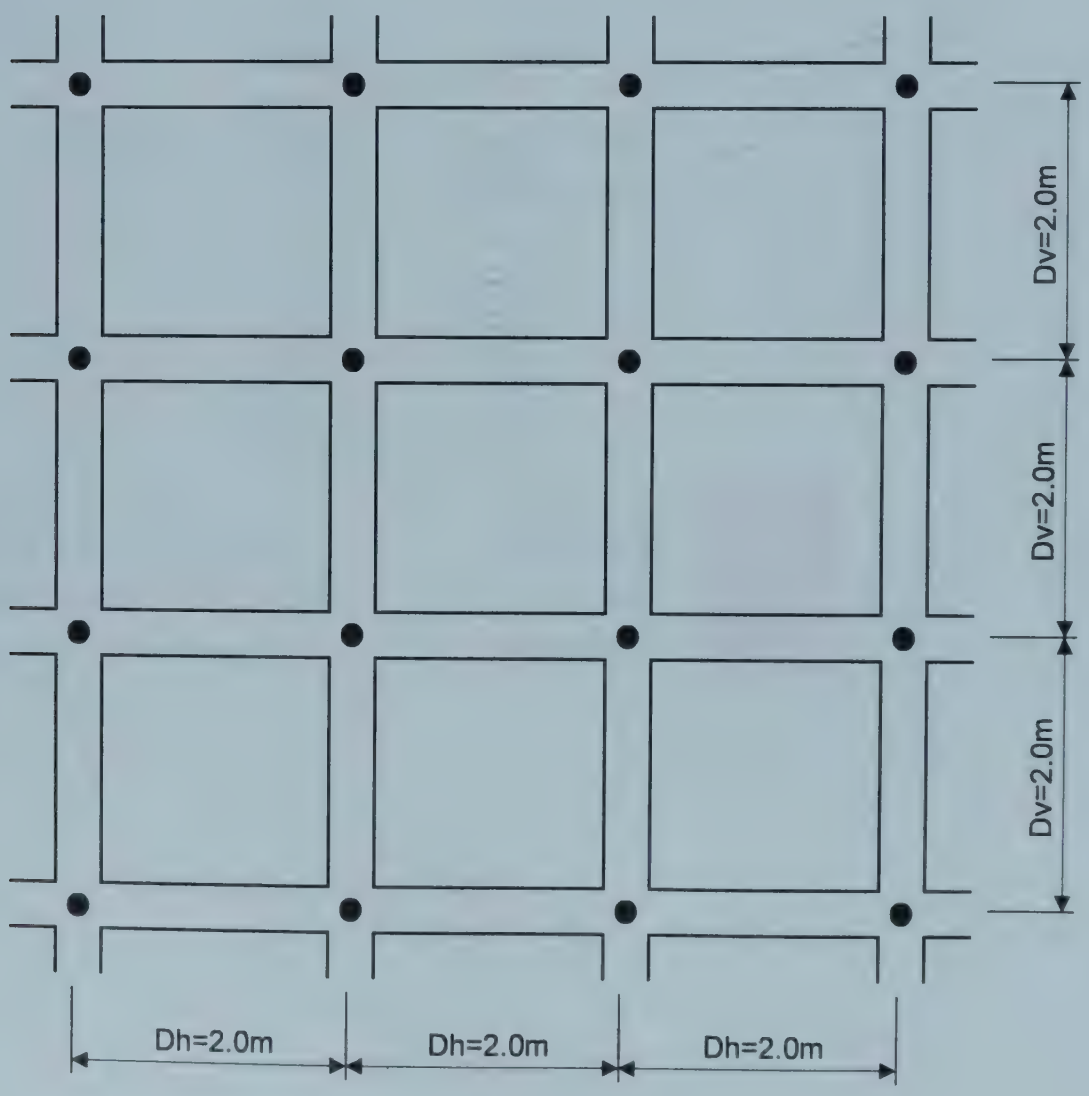
Veight, sectional area, and peripheral length of strange appearance steel ba

Alias	Unit mass	lominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	32.4	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{32.4}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 8.76 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 8.76 \times 2.00^2$$

$$= 4.38 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 8.76 \times 2.00$$

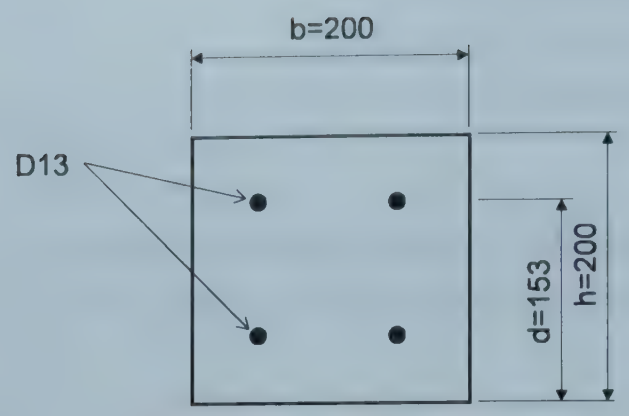
$$= 8.76 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13 ×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{4.38 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 2.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{4.38 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 81.2 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{8.76 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.14 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{8.76 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.51 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Slide 2-Slip Surface 2 (Zone 4-2)

Design load　Td=　27.6 kN／No.　(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reintocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar　D13 ×上下各　2 本

126.7

Calculation result

Uniformly-distributed load	w=	7.46	kN/m
Maximum bending moment	M=	3.73	kN･m
Maximum shear strength	S=	7.46	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	1.8	N/mm2	OK	6
◎Tensile stress of reinforcing bar	σs=	69.1	N/mm2	OK	180
◎Shear stress of mortar	τc=	0.12	N/mm2	OK	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.22	N/mm2	OK	1.4

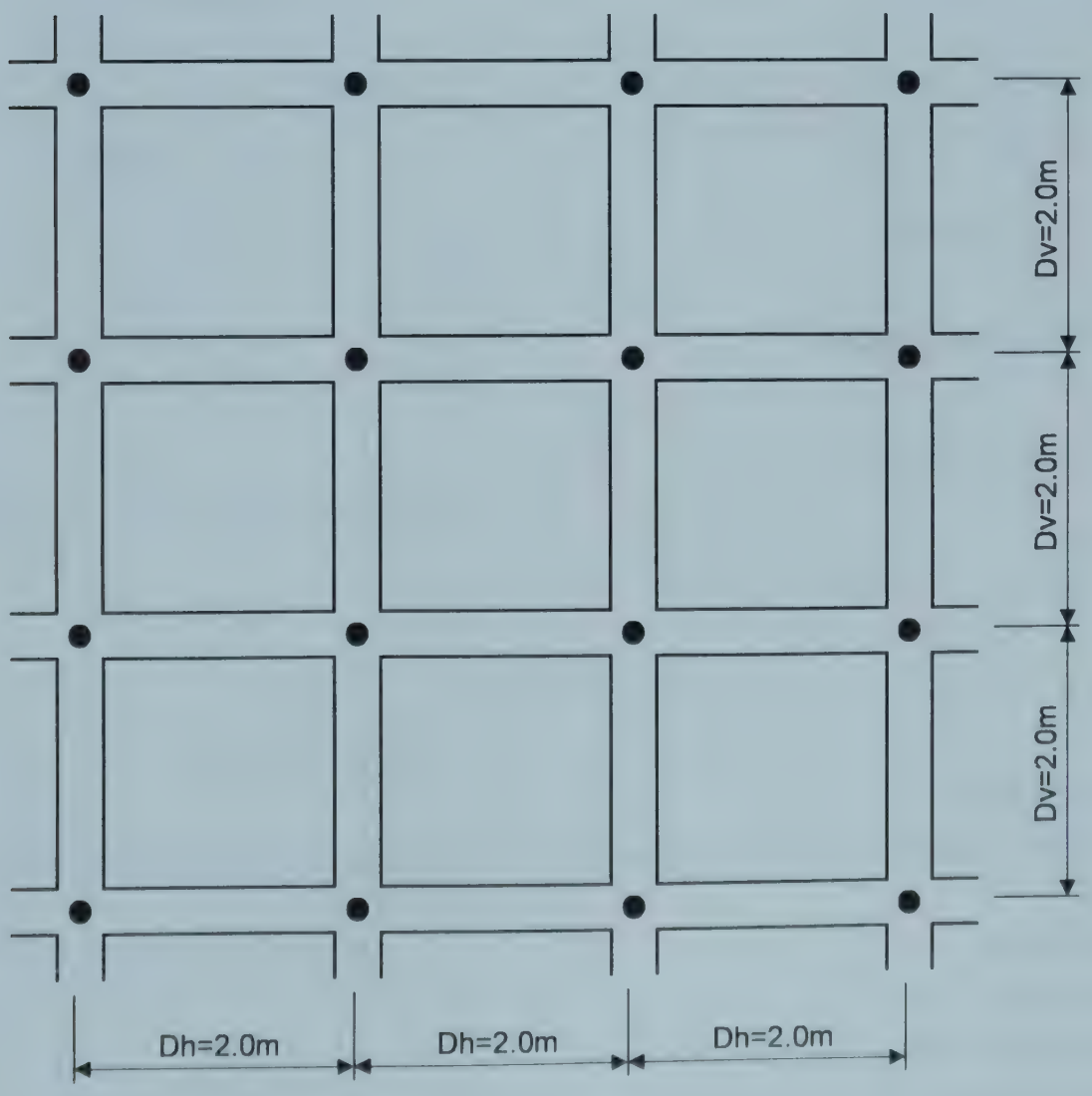
Weight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	27.6	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{27.6}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 7.46 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 7.46 \times 2.00^2$$

$$= 3.73 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 7.46 \times 2.00$$

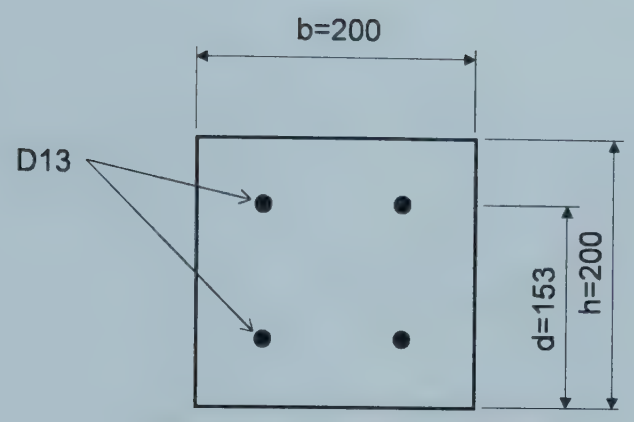
$$= 7.46 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13 ×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{3.73 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 1.8 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{3.73 \times \#^{5}}{300 \times 235^{2}} \times 307.11$$

$$= 69.1 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$Tc = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{7.46 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.12 \ N/mm^2 < T_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$Tc = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{7.46 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.44 \ N/mm^2 < T_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Slide 2-Slip Surface 3 (Zone 3 and Zone 4-2)

Design load	Td=	32.4 kN／No.	(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reinfocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar		D13	×上下各 2 本
		126.7	

Calculation result

Uniformly-distributed load	w=	8.76	kN/m
Maximum bending moment	M=	4.38	kN･m
Maximum shear strength	S=	8.76	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	2.1	N/mm2	OK	6
◎Tensile stress of reinforcing bar	σs=	81.2	N/mm2	OK	180
◎Shear stress of mortar	τc=	0.14	N/mm2	OK	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.26	N/mm2	OK	1.4

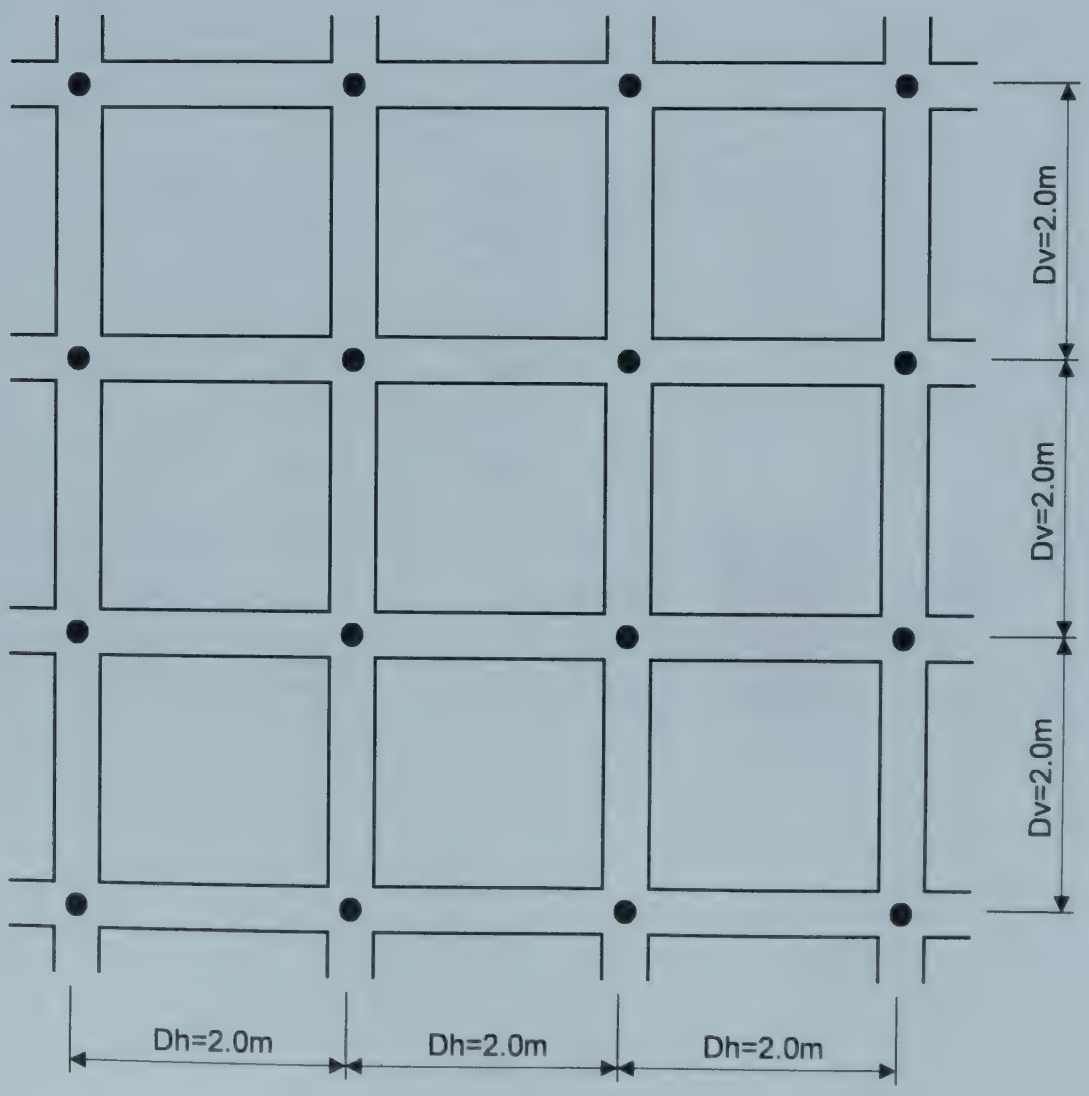
Veight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	lominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	32.4	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{32.4}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 8.76 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 8.76 \times 2.00^2$$

$$= 4.38 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 8.76 \times 2.00$$

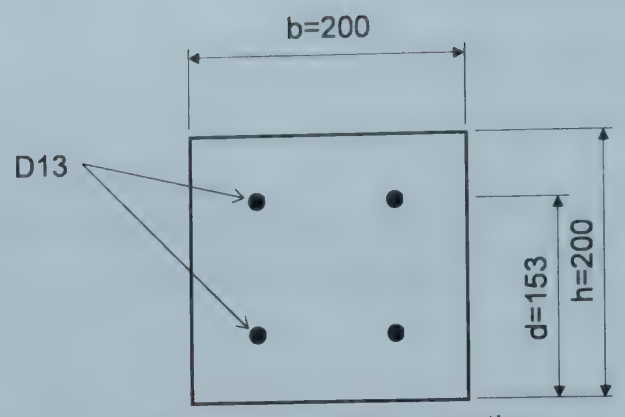
$$= 8.76 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300	mm
Head space of the frame	d	=	235	mm
Reinforcing bar	D13	×	Upper and lov	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{4.38 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 2.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{4.38 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 81.2 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{8.76 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.14 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{8.76 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.51 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Loosen Slope A (Zone 7)

Loosen slope A

Design load

Td=	32.7	kN／No.	(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reintocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar: D13 ×上下各 2 本

126.7

Calculation result

Uniformly-distributed load	w=	8.84	kN/m
Maximum bending moment	M=	4.42	kN･m
Maximum shear strength	S=	8.84	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	2.1	N/mm2	O K	6
◎Tensile stress of reinforcing bar	σs=	81.9	N/mm2	O K	180
◎Shear stress of mortar	τc=	0.14	N/mm2	O K	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.26	N/mm2	O K	1.4

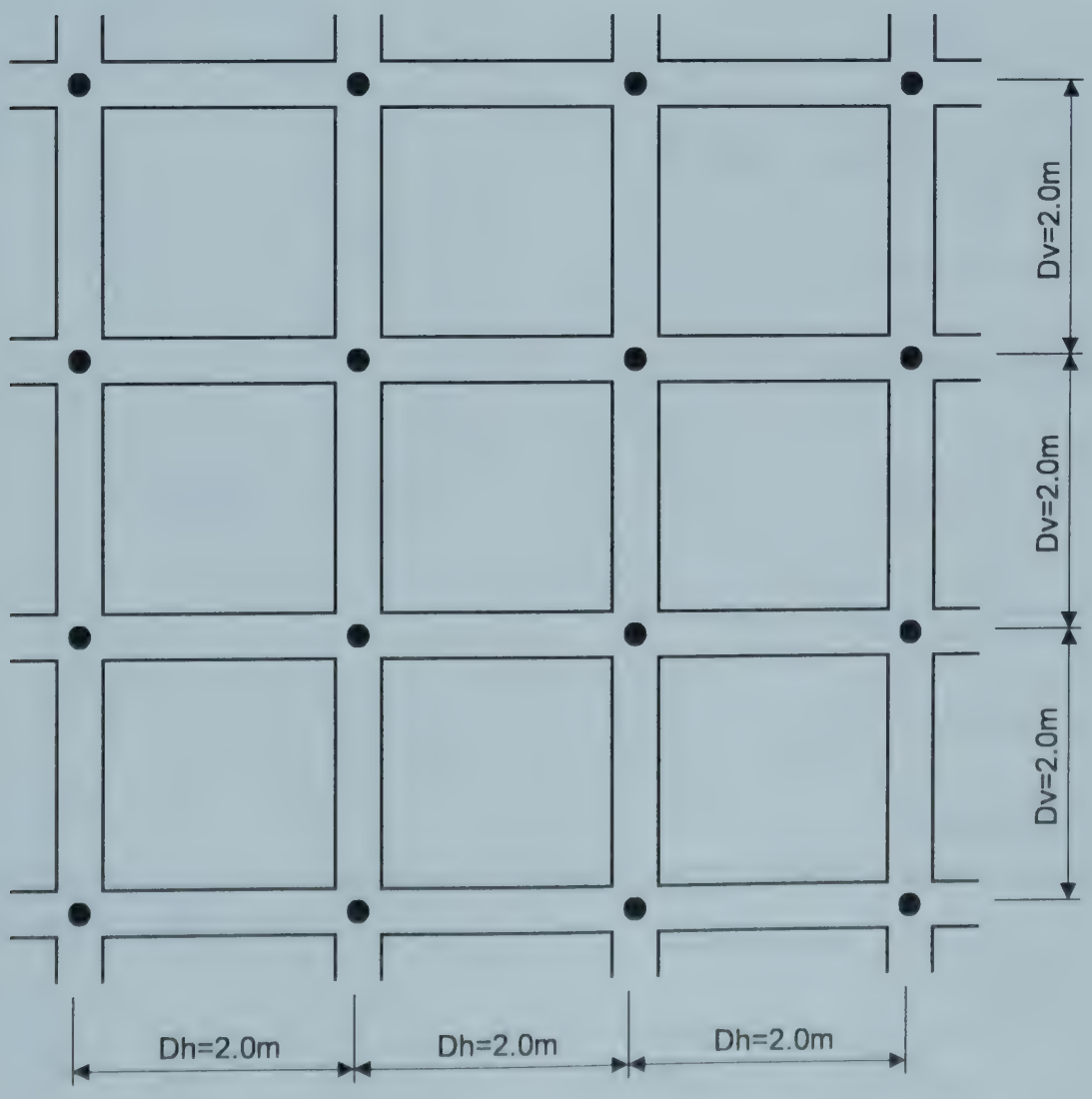
Veight, sectional area, and peripheral length of strange appearance steel b:

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	32.7	kN／No.
Span of the transversal frame	Lx	=	2.00	m
Span of the longitudinal frame	Ly	=	2.00	m
Transverse space of the reinforcing material	Dh	=	2.00	m
Longitudinal space of the reinforcing material	Dv	=	2.00	m
Width of the frame	b	=	300	mm
Hight of the frame	h	=	300	mm
Head space of the frame	d	=	235	mm
Design strength of the mortar	σ_{ck}	=	18	N/mm^2
Allowable compressive stress	σ_{ca}	=	6	N/mm^2
Allowable shear stress	τ_{ca}	=	0.40	N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4	N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180	N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{32.7}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 8.84 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 8.84 \times 2.00^2$$

$$= 4.42 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 8.84 \times 2.00$$

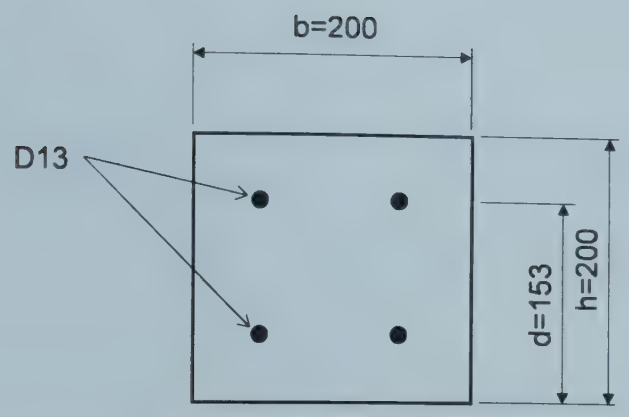
$$= 8.84 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13 ×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{4.42 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 2.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{4.42 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 81.9 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$Tc = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{8.84 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.14 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$Tc = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{8.84 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.52 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Calculation of Crib Works

Loosen Slope B (Zone 6)

A02-153

Loosen slope B

Design load	Td=	32.7	kN／No.	(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reinfocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar: D13 ×上下各 2 本

126.7

Calculation result

Uniformly-distributed load	w=	8.84	kN/m
Maximum bending moment	M=	4.42	kN･m
Maximum shear strength	S=	8.84	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	2.1	N/mm2	O K	6
◎Tensile stress of reinforcing bar	σs=	81.9	N/mm2	O K	180
◎Shear stress of mortar	τc=	0.14	N/mm2	O K	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.26	N/mm2	O K	1.4

Veight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	lominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
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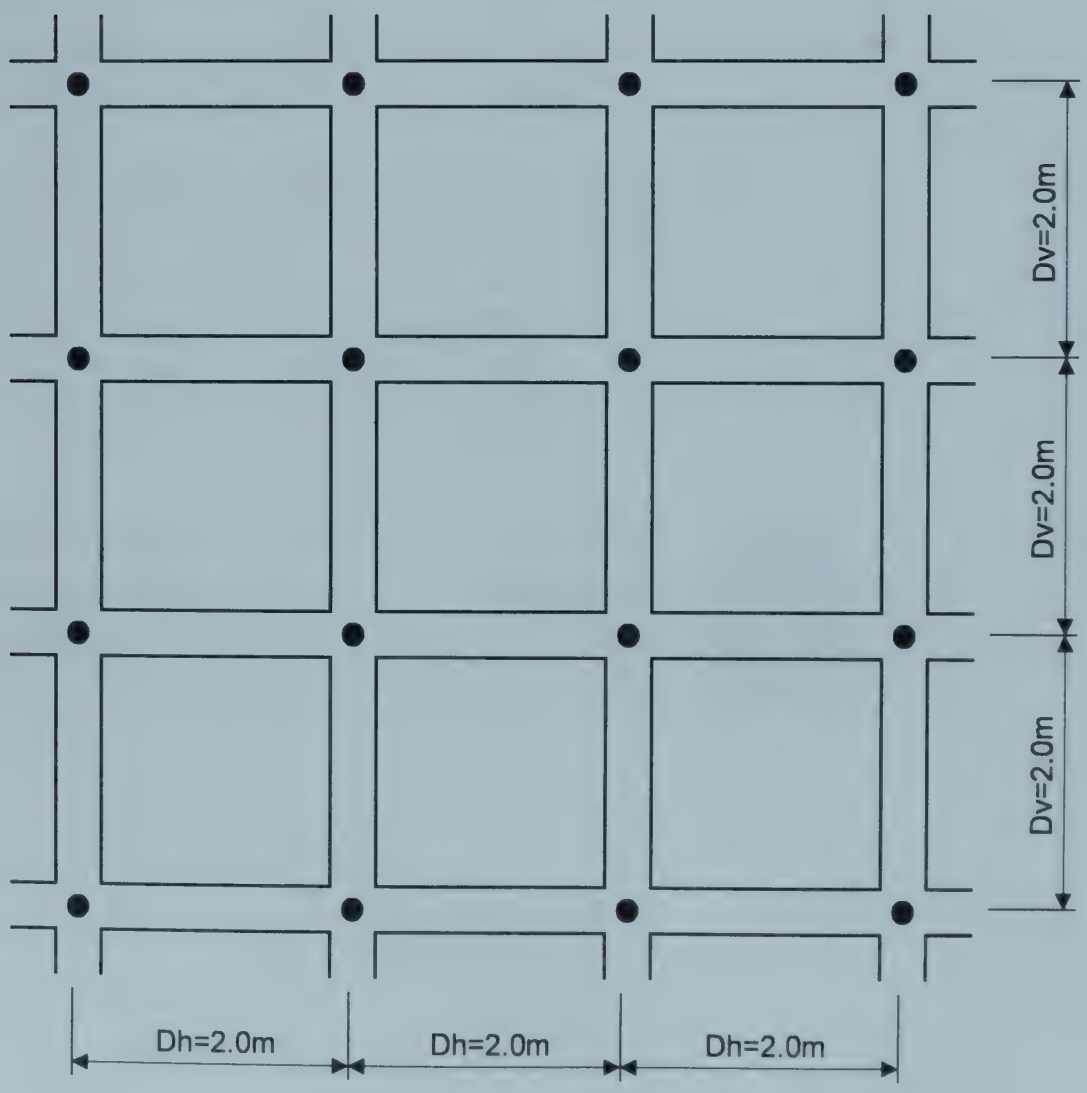
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	32.7 kN／No.
Span of the transversal frame	Lx	=	2.00 m
Span of the longitudinal frame	Ly	=	2.00 m
Transverse space of the reinforcing material	Dh	=	2.00 m
Longitudinal space of the reinforcing material	Dv	=	2.00 m
Width of the frame	b	=	300 mm
Hight of the frame	h	=	300 mm
Head space of the frame	d	=	235 mm
Design strength of the mortar	σ_{ck}	=	18 N/mm^2
Allowable compressive stress	σ_{ca}	=	6 N/mm^2
Allowable shear stress	τ_{ca}	=	0.40 N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4 N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180 N/mm^2



Layout drawing of reference chart 1(crib work)

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Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{32.7}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 8.84 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 8.84 \times 2.00^2$$

$$= 4.42 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 8.84 \times 2.00$$

$$= 8.84 \text{ kN}$$

④ Check of stress degree

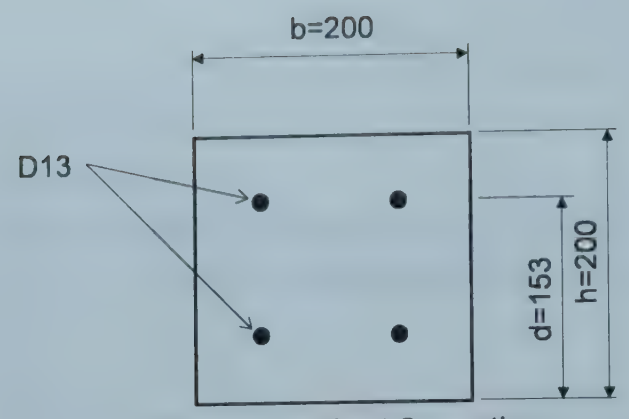
The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame b = 300 mm

Head space of the frame d = 235 mm

Reinforcing bar D13 × Upper and low 2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{4.42 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 2.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{4.42 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 81.9 \ N/mm^2 < \sigma_{sa} = 180 \ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{8.84 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.14 \ N/mm^2 < \tau_{ca} = 0.40 \ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \ mm \times 2$ Pieces

$$= \frac{8.84 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.52 \ N/mm^2 < \tau_{oa} = 1.4 \ N/mm^2 \quad OK$$

Appendix 1.4

Drawings

(Plan View, Representative Cross sections and

Representative drawing of crib works

Appendix 1.4

Drawings

(Plan View, Representative Cross sections and

Representative drawing of crib works

A02-162

Horizontal drainage borings

Concrete crib works

Concrete crib works with rock bolt

Drainage ditch

Loosen slope A

Loosen slope B

Slide -1

BV1-1

Concrete crib works with rock bolt

Drainage ditch

Drainage ditch

Horizontal drainage borings

Slide -2

Concrete crib works with rock bolt

BV2-1

Concrete crib works

Concrete crib works with rock bolt

Concrete crib works

560.000

550.000

540.000

530.000

520.000

510.000

500.000

490.000

480.000

470.000

TP

EP

KM

N

0 50m

Slide-1 & 2

horizontal drainage borings:

L=30m,25m, Angle of drilling =10degrees

Concrete crib work:

Cross section=0.3m*0.3m, Interval=2.00m*2.00m

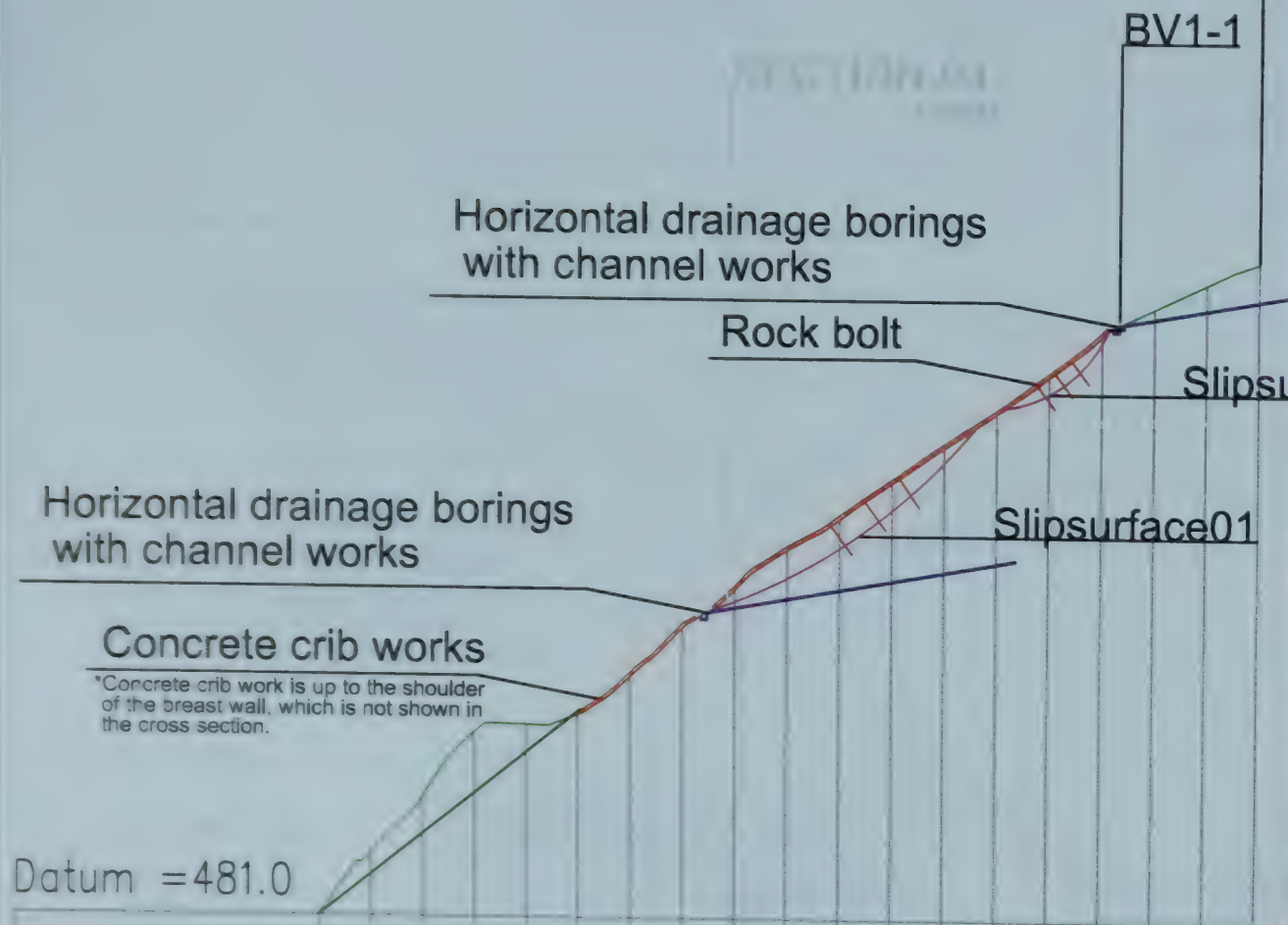
Filling material between beems: Vegetation

Rock bolt:

σsa=200N/mm2(SD345), D=25mm

Slide-1

SECTION-03



SECTION-03

Rock bolt (Slipsurface01):
3Steps, Interval(H)=4.0m, Interval(SL)=4.0m, L=3.4(+0.3)m
σ sa=200N/mm^2 (SD345), D=25mm

Rock bolt (Slipsurface02):
3Steps, Interval(H)=4.0m, Interval(SL)=2.0m, L=2.7(+0.3)m
σ sa=200N/mm^2 (SD345), D=25mm

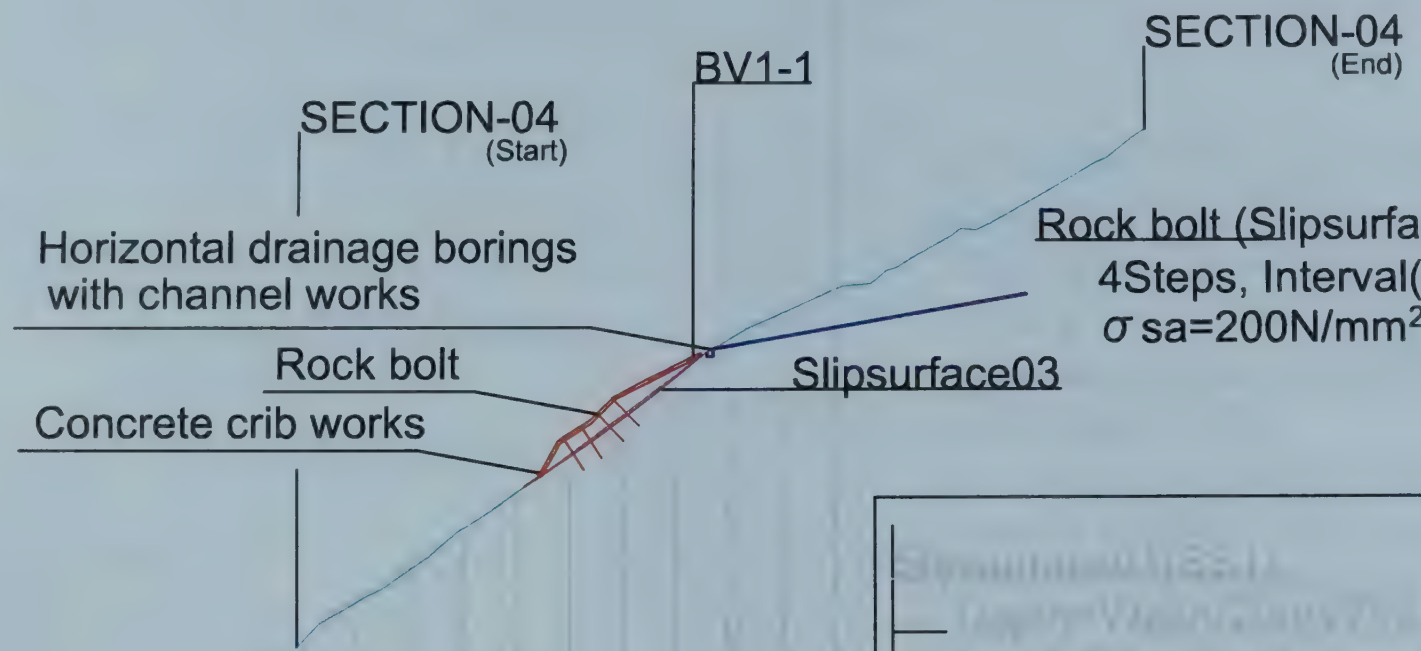
Slipsurface01(SS1):
- Depth=Width(20m)/7=2.857m
- Head & Toe: Knick point
- Circular slip, Average tilt angle of slipsurface α =38 degrees
- Internal friction angle Φ'= α =38 degrees
- c'= 0.10kN/m^2 (refer to SS2) *Backanalysis 0.68 kN/m^2 (Fs=1.0)

Slipsurface02(SS2):
- Depth=Width(11m)/7=1.5714m
- Head & Toe: Knick point
- Circular slip, Average tilt angle of slipsurface α =38 degrees
- Internal friction angle Φ'= α =38 degrees
- Backanalysis c'= 0.10 kN/m^2 (Fs=1.0)

A02-163

Slide-1

SECTION-04

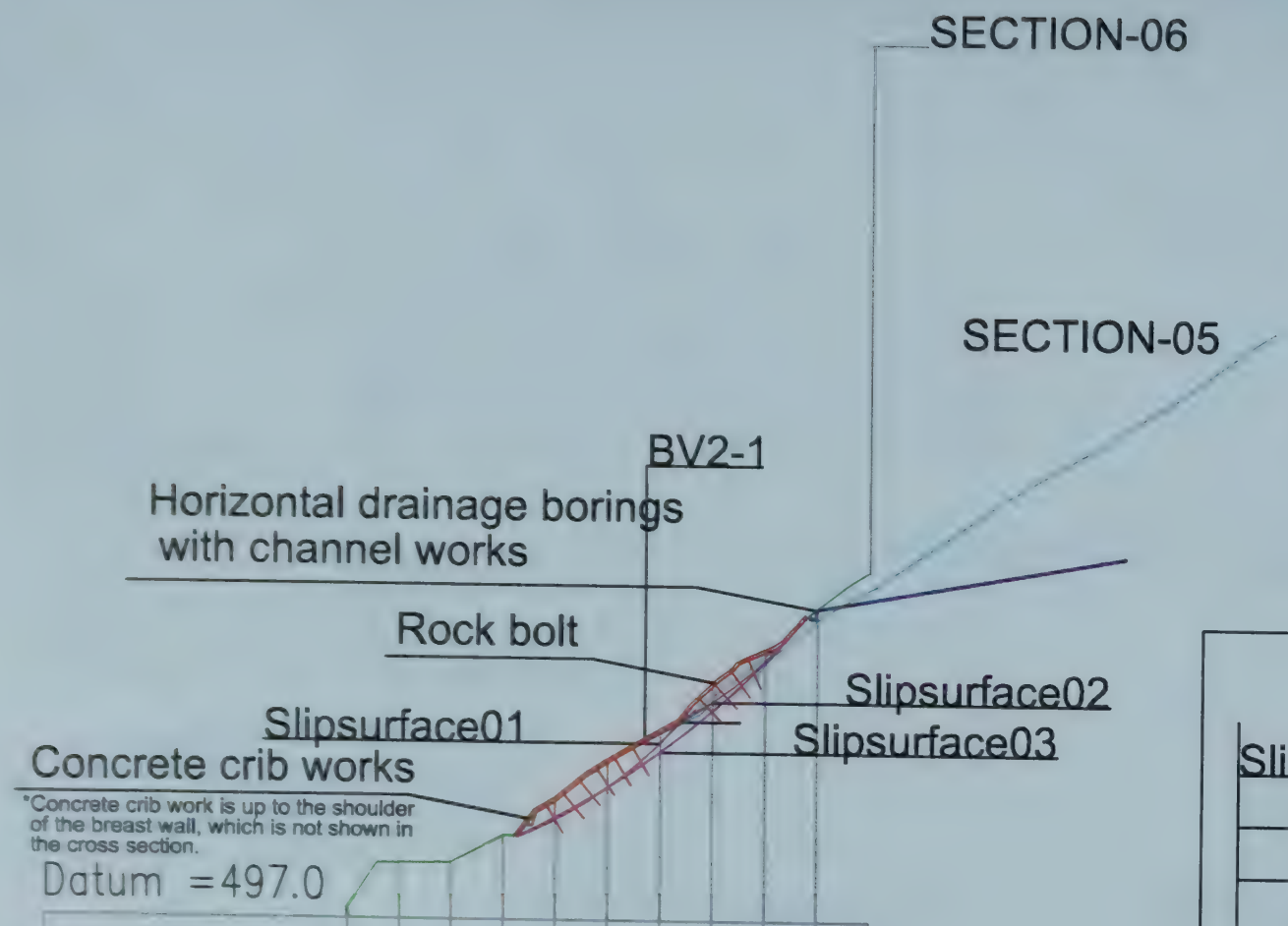


Rock bolt (Slipsurface03):
4Steps, Interval(H)=4.0m, Interval(SL)=2.0m, L=3.3(+0.3)m
σsa=200N/mm² (SD345), D=25mm

Slipsurface03(SS3):
- Head & Toe: Knick point
- Circular slip, Average tilt angle of slipsurface α =37 degrees
- Internal friction angle Φ'= α =37 degrees
- Backanalysis c'= 0.00 kN/m (Fs=1.0)

A02-164

Slide-2



Existing Gr.Level	498.000	502.371	502.459	505.014	508.400	512.167	515.070	519.505	522.866	527.048
Chainage (m)	0.000	5.000	10.000	15.000	20.000	25.000	30.000	35.000	40.000	45.000

SECTION-06

Rock bolt (Slipsurface01):
6Steps, Interval(H)=2.0m, Interval(SL)=2.0m, L=3.0(+0.3)m
σ sa=200N/mm^2 (SD345), D=25mm

Rock bolt (Slipsurface02):
4Steps, Interval(H)=2.0m, Interval(SL)=2.0m, L=3.0(+0.3)m
σ sa=200N/mm^2 (SD345), D=25mm

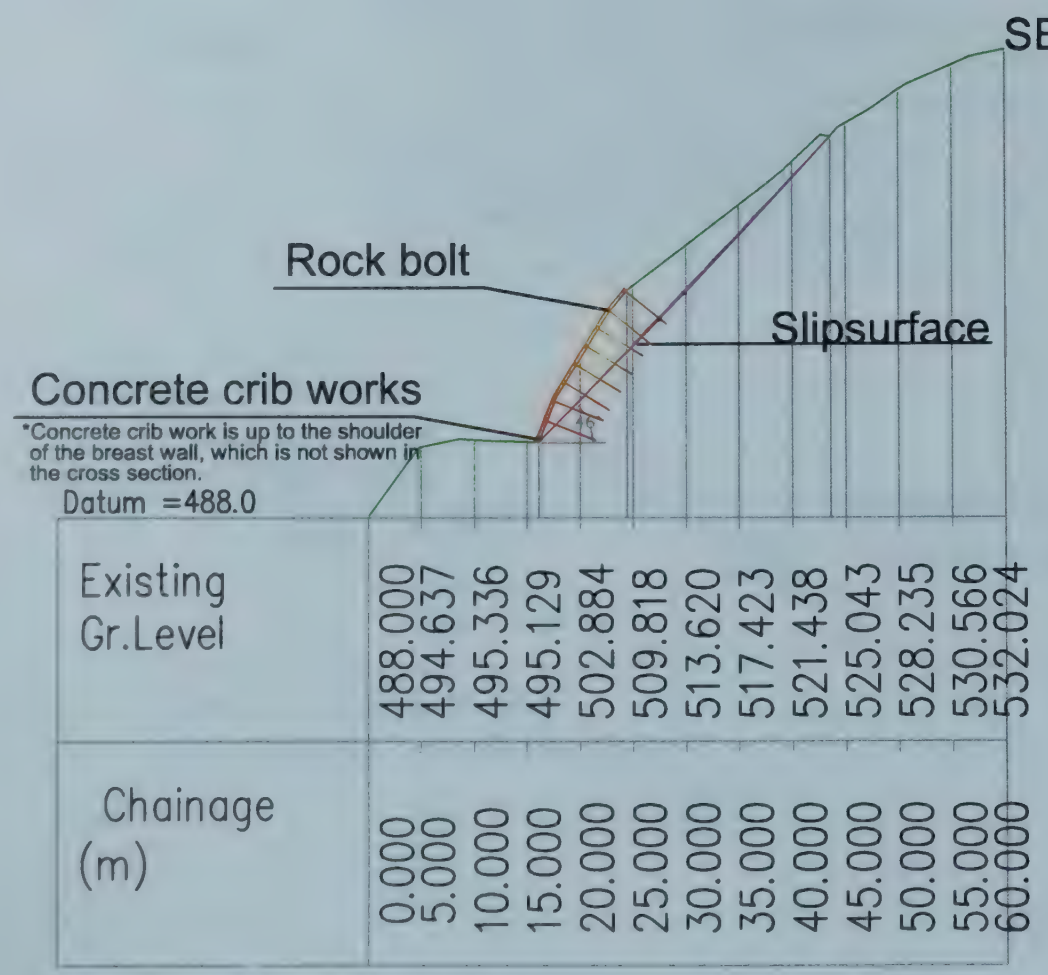
Rock bolt (Slipsurface03):
10Steps, Interval(H)=2.0m, Interval(SL)=2.0m, L=3.0(+0.3)m
σ sa=200N/mm^2 (SD345), D=25mm

Slipsurface01(SS1):
- Head, Toe & cliff top: Knick point
- Circular slip, Average tilt angle of slipsurface α =36 degrees
- Internal friction angle Φ'= α =36 degrees
- Backanalysis c'= 0.92 kN/m^2 (Fs=1.0)

Slipsurface02(SS2):
- Head, Toe & cliff top: Knick point
- Circular slip, Average tilt angle of slipsurface α =38 degrees
- Internal friction angle Φ'= α =36 degrees(refer to SS1)
- c'= 0.92kN/m^2 (refer to SS1) *Backanalysis 1.21 kN/m^2 (Fs=1.0)

Slipsurface03(SS3):
- Head, Toe & cliff top: Knick point
- Circular slip, Average tilt angle of slipsurface α =37 degrees
- Internal friction angle Φ'= α =36 degrees(refer to SS1)
- c'= 0.92 kN/m^2 (refer to SS1)

Loosen slope A



SECTION-01

Rrock bolt:

8Steps, Interval(H)=8.0m, Interval(SL)=8.0m, L=4.9(+0.3)m

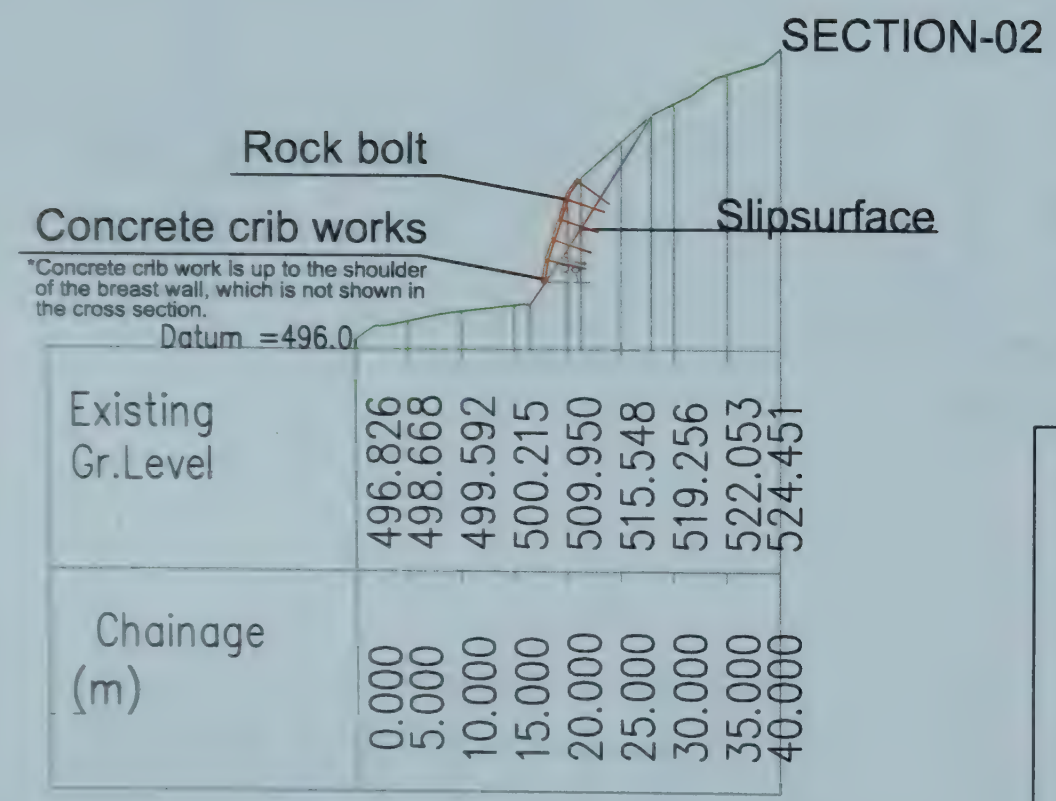
σ sa=200N/mm^2 (SD345), D=25mm

Slipsurface:

- Head & Toe: Knick point
- Circular slip, Average tilt angle of slipsurface α =46 degrees
- Internal friction angle Φ'= α =38 degrees(refer to Slide-1 SS1)
- Backanalysis c'= 7.21 kN/m^2 (Fs=1.0)

A02-166

Loosen slope B



SECTION-02

Rrock bolt:

5Steps, Interval(H)=2.0m, Interval(SL)=2.0m, L=3.6(+0.3)m

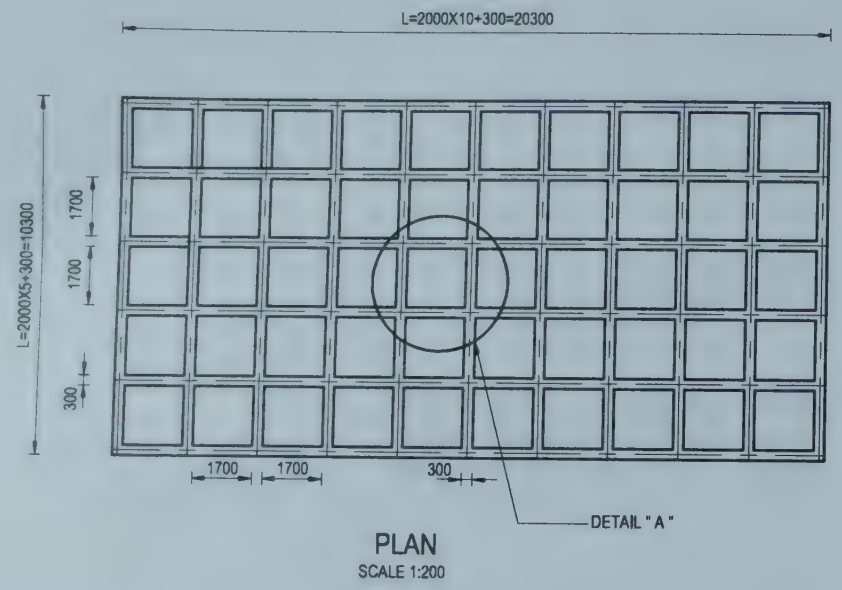
σ sa=200N/mm^2 (SD345), D=25mm

Slipsurface:

- Head & Toe: Knick point
- Circular slip, Average tilt angle of slipsurface α =58 degrees
- Internal friction angle Φ'= α =38 degrees(refer to Slide-1 SS1)
- Backanalysis c'= 9.93 kN/m^2 (Fs=1.0)

A02-167

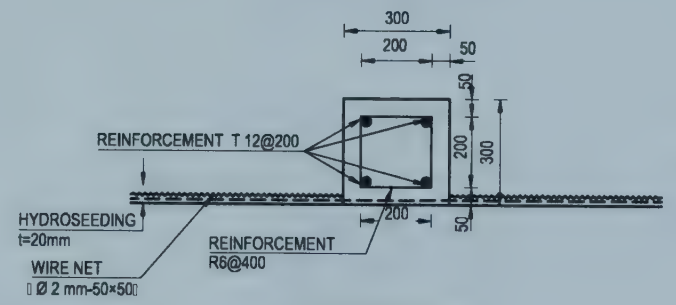
Representative drawing of crib works



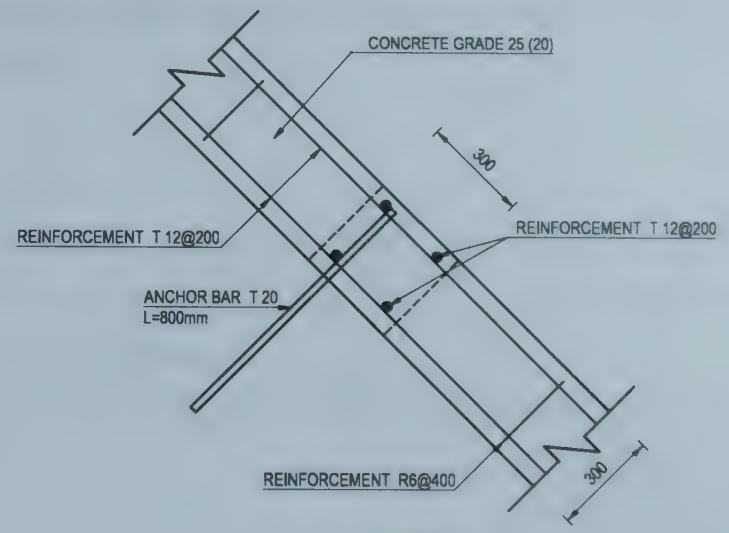
PLAN
SCALE 1:200

QUANTITY CALCULATIONS FOR CRIB WORK AREA OF 210 m² (20.3 m x 10.3 m)

TYPE		UNIT	QUANTITY	CALCULATION FORMULA (Kg)	
FRAME LENGTH		m	215.3	10.3x11+1.7x10x6	
ANCHOR BAR (T20, 800 mm)		nos	66.0 215.30	11x6	66.00
ANCHOR PIN (T12 500mm)		nos	120.0	2x(10x6)	120.00
REINFORCEMENT WEIGHT	T	t	0.84	20.3x6x4x0.888	432.63
				10.3x11x4x0.888	402.44
	R		0.11	0.90x5x55x0.222	54.95
				0.90x5x60x0.222	59.94
				TOTAL	949.96
CONCRETE GRADE 25(20)		m^3	19.4	215.3x0.30x0.30	19.38
AREA WITHIN THE CRIB		m^2	144.5	1.70x1.70x50	144.50

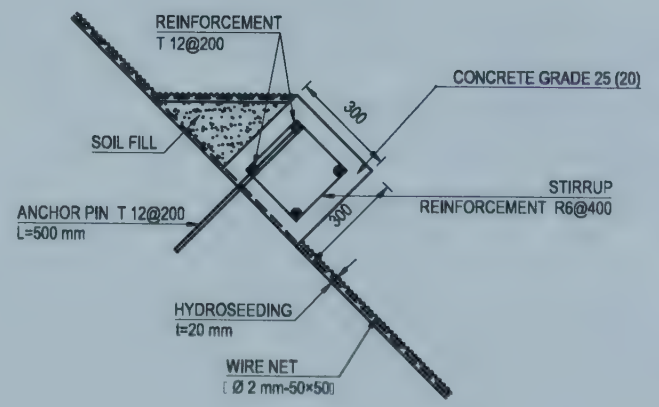


SECTION A - A
SCALE 1:20

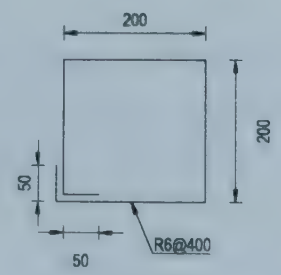


SECTION C - C
SCALE 1:20

DETAIL " A "
SCALE 1 :40



SECTION B - B
SCALE 1:20



STIRRUP DETAIL
SCALE 1:10

NOTE
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
2. ALL QUANTITIES GIVEN IN THE TABLE ARE ONLY FOR REFERENCE USE

A02-168

Design for Slide No.3

Appendix 2.1 Back Analysis Results

Appendix 2.2 Calculation of Rock Bolt

Appendix 2.3 Calculation of Crib Works

Appendix 2.4 Calculation of Ground Anchor

Appendix 2.5 Drawings (Plan View, Representative Cross sections, Representative drawing of crib works and Representative drawing of ground anchor pillow and anchor plate)

Appendix 2.1

Back Analysis Results

Back Analysis Result

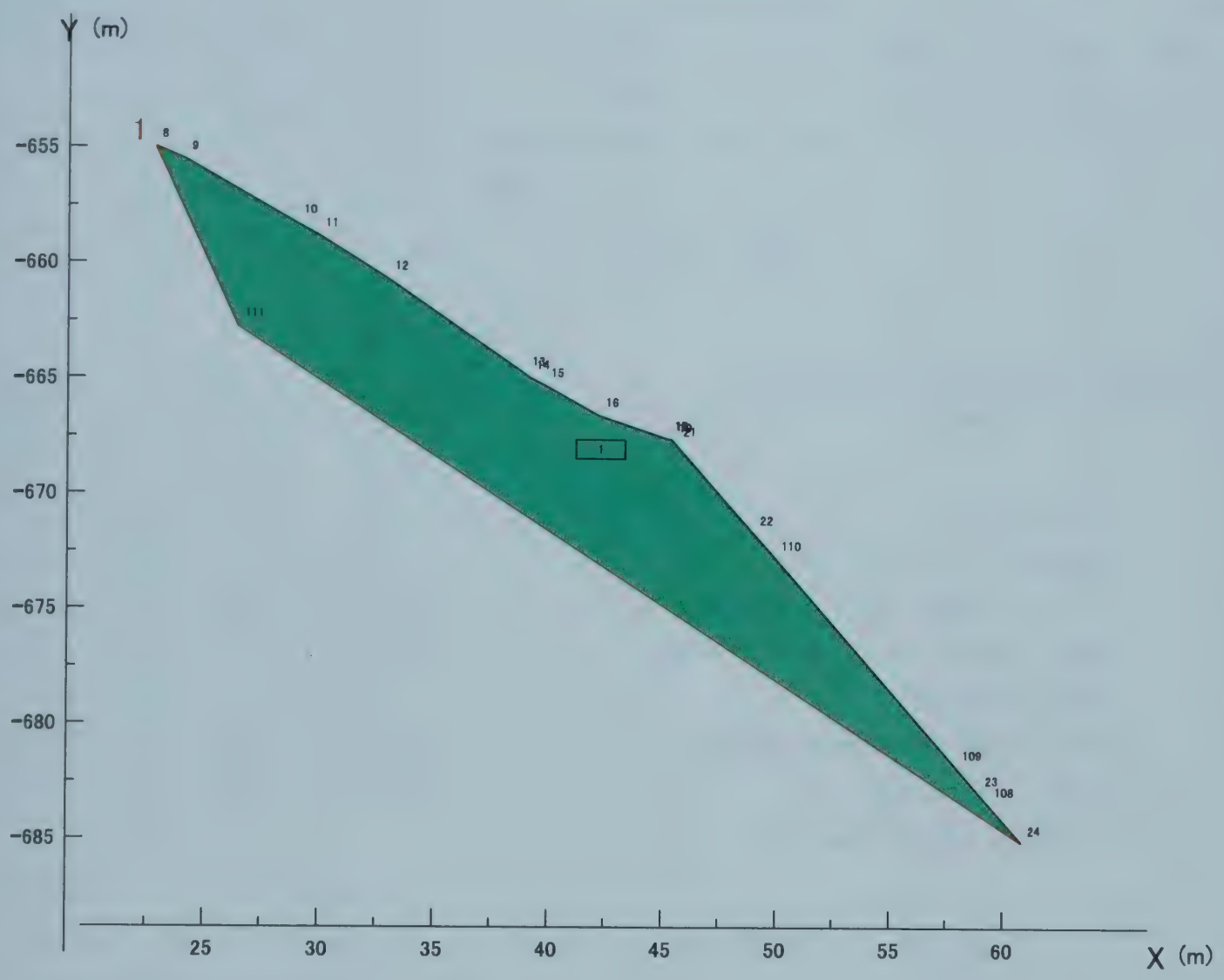
Slide 3

Back Analysis Result

Slide 3 - Upper 2)-1

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title	.....	Slide 3
Analysis type	.....	Inverse analysis
Req. for safety factor (Ordinary)	.....	1.000
Req. for safety factor (Seismic)	.....	1.000
Calculation method	.....	Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{\sum W*\sin(a)}$$

Fs	:	Safety factor
C	:	Cohesion (kN/m²)
L	:	Circle length of a slice (m)
W	:	Total weight of a slice (kN/m)
a	:	Angle (degree)
U	:	Pore water pressure (kN/m²)
b	:	Width of a slice (m)
PHI	:	Friction angle (degree)

Width of a slice	.....	10.0
Load condition	.....	0 (= 0 Affects both resisting and sliding forces)
		(= 1 Only sliding force)

(DATA 03.)　// NODAL POINT //

No.	Coordinate	
	X (m)	Y (m)
8	22.77	-654.96
9	24.06	-655.50
10	28.92	-658.22
11	29.89	-658.77
12	32.91	-660.62
13	38.92	-664.75
14	39.12	-664.89
15	39.77	-665.24
16	42.16	-666.52
17	45.17	-667.51
18	45.20	-667.52
19	45.36	-667.57
21	45.54	-667.77
22	48.92	-671.59
23	58.92	-682.92
24	60.83	-685.08
108	59.34	-683.39
109	57.91	-681.77
110	49.88	-672.68
111	26.40	-662.74

(DATA 05.)　// LAYER //

No.	Material property No.	Num of node	Node numbers composing a layer											
1	1	20	8	111	24	108	23	109	110	22	21	19	18	17
			16	15	14	13	12	11	10	9				

(DATA 06.1)　// MATERIAL PROPERTIES //

No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Cohesion (kN/m^2)	rate of increase of cohesion	Friction angle (Degree)	Tau (kN/m^2)
1	18.00	18.00	------	-------	--------	-----

The layer for Inverse analysis　1

Y-Coordinate of base line of cohesion (m) =　0.00

3. Inverse Analysis

INVERSE ANALYSIS LIST

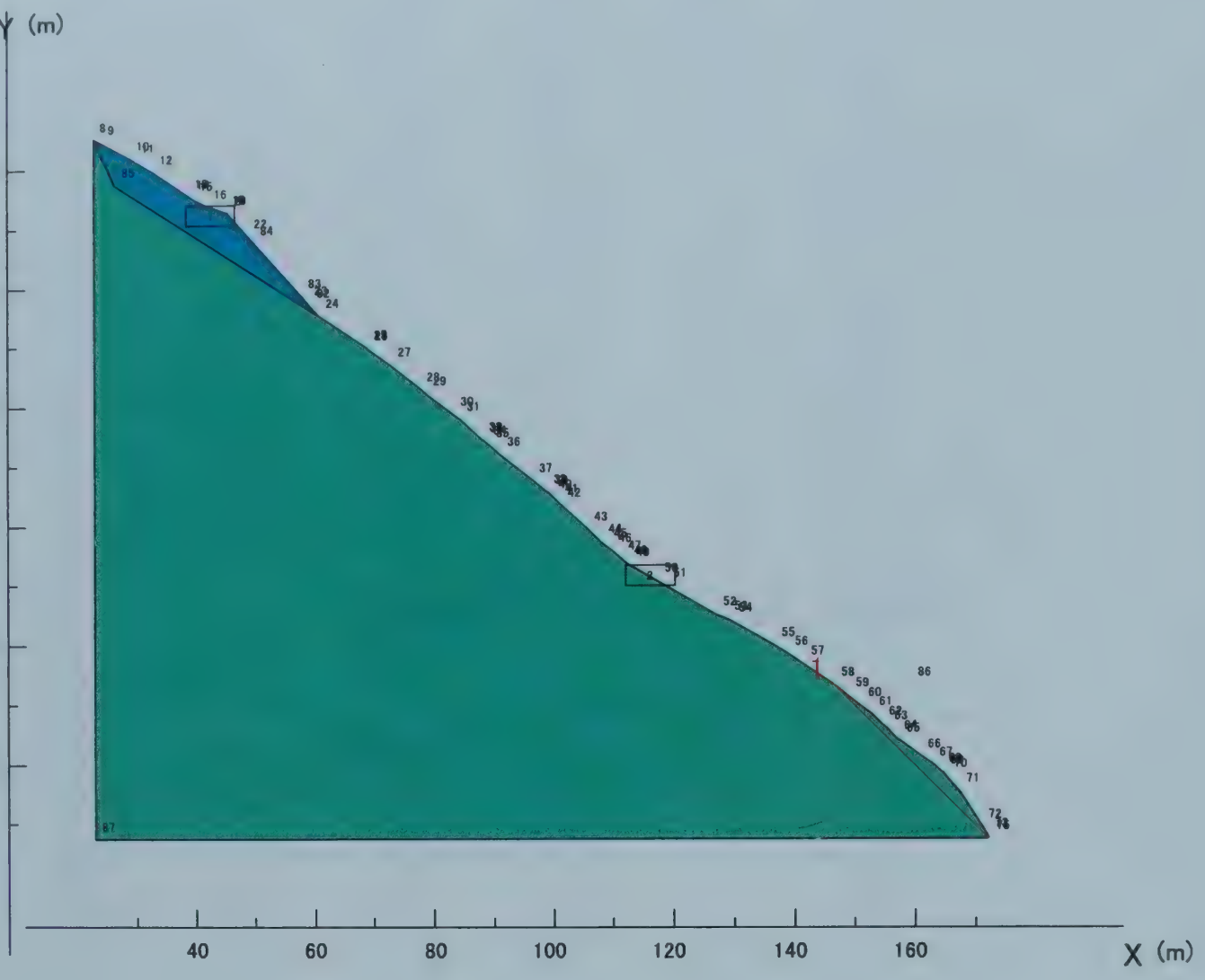
Ordinary condition					
Cohesion (kN/m²)	Angle (Degree)	tan(PHI)	Angle (Degree)	tan(PHI)	Cohesion (kN/m²)
0.0	34.76	0.69	0.00	0.00	38.77
1.0	34.06	0.68	1.00	0.02	37.79
2.0	33.35	0.66	2.00	0.03	36.82
3.0	32.63	0.64	3.00	0.05	35.84
4.0	31.89	0.62	4.00	0.07	34.86
5.0	31.15	0.60	5.00	0.09	33.88
6.0	30.39	0.59	6.00	0.11	32.90
7.0	29.62	0.57	7.00	0.12	31.91
8.0	28.84	0.55	8.00	0.14	30.92
9.0	28.05	0.53	9.00	0.16	29.92
10.0	27.24	0.51	10.00	0.18	28.92
11.0	26.43	0.50	11.00	0.19	27.91
12.0	25.60	0.48	12.00	0.21	26.89
13.0	24.76	0.46	13.00	0.23	25.87
14.0	23.91	0.44	14.00	0.25	24.84
15.0	23.05	0.43	15.00	0.27	23.80
16.0	22.17	0.41	16.00	0.29	22.75
17.0	21.29	0.39	17.00	0.31	21.69
18.0	20.39	0.37	18.00	0.32	20.61
19.0	19.48	0.35	19.00	0.34	19.53
20.0	18.57	0.34	20.00	0.36	18.43
21.0	17.64	0.32	21.00	0.38	17.32
22.0	16.71	0.30	22.00	0.40	16.19
23.0	15.76	0.28	23.00	0.42	15.05
24.0	14.81	0.26	24.00	0.45	13.89
25.0	13.84	0.25	25.00	0.47	12.72
26.0	12.87	0.23	26.00	0.49	11.52
27.0	11.89	0.21	27.00	0.51	10.30
28.0	10.91	0.19	28.00	0.53	9.06
29.0	9.92	0.17	29.00	0.55	7.80
30.0	8.92	0.16	30.00	0.58	6.51
31.0	7.92	0.14	31.00	0.60	5.20
32.0	6.91	0.12	32.00	0.62	3.86
33.0	5.90	0.10	33.00	0.65	2.48
34.0	4.88	0.09	34.00	0.67	1.08
35.0	3.86	0.07	35.00	0.70	0.00
36.0	2.84	0.05	36.00	0.73	0.00
37.0	1.81	0.03	37.00	0.75	0.00
38.0	0.79	0.01	38.00	0.78	0.00
39.0	0.00	0.00	39.00	0.81	0.00
40.0	0.00	0.00	40.00	0.84	0.00
41.0	0.00	0.00	41.00	0.87	0.00
42.0	0.00	0.00	42.00	0.90	0.00
43.0	0.00	0.00	43.00	0.93	0.00
44.0	0.00	0.00	44.00	0.97	0.00
45.0	0.00	0.00	45.00	1.00	0.00
46.0	0.00	0.00	46.00	1.04	0.00
47.0	0.00	0.00	47.00	1.07	0.00
48.0	0.00	0.00	48.00	1.11	0.00
49.0	0.00	0.00	49.00	1.15	0.00
50.0	0.00	0.00	50.00	1.19	0.00

Back Analysis Result

Slide 3 - Lower 2)-2

1. Input data list

// Design section //



(DATA 01.) // ANALYSIS //

Title .....
Analysis type ..... Inverse analysis
Req. for safety factor (Ordinary) 1.000
Req. for safety factor (Seismic) 1.000
Calculation method ..... Road public works -Guideline for cutting ground stability (Effective stress)

Expression

$$F_s = \frac{\sum[C*L+(W*\cos(a)-U*b*\cos(a))*\tan(PHI)]}{\sum W*\sin(a)}$$

Fs : Safety factor
C : Cohesion (kN/m^2)
L : Circle length of a slice (m)
W : Total weight of a slice (kN/m)
a : Angle (degree)
U : Pore water pressure (kN/m^2)
b : Width of a slice (m)
PHI : Friction angle (degree)

Width of a slice 10.0
Load condition ..... 0 (= 0 Affects both resisting and sliding forces)
(= 1 Only sliding force)

(DATA 06.1) // MATERIAL PROPERTIES //

No.	Saturated weight (kN/m³)	Wet weight (kN/m³)	Cohesion (kN/m²)	rate of increase of cohesion	Friction angle (Degree)	Tau (kN/m²)
1	18.00	18.00	2.48	0.00	33.00	0.0
2	18.00	18.00	------	-------	--------	-----

The layer for Inverse analysis 2

Y-Coordinate of base line of cohesion (m) = 0.00

(DATA 07.) // SLIP SURFACE //

Slip surface No. = 1

NODE NO.	1	2
X (m)	146.61	172.22
Y (m)	-746.98	-772.46
R (m)	0.00	0.00
FIXED X (m)	0.00	0.00
FIXED Y (m)	0.00	0.00

3. Inverse Analysis

INVERSE ANALYSIS LIST

Ordinary condition					
Cohesion (kN/m²)	Angle (Degree)	tan(PHI)	Angle (Degree)	tan(PHI)	Cohesion (kN/m²)
0.0	44.85	0.99	0.00	0.00	15.02
1.0	42.88	0.93	1.00	0.02	14.76
2.0	40.78	0.86	2.00	0.03	14.49
3.0	38.53	0.80	3.00	0.05	14.23
4.0	36.13	0.73	4.00	0.07	13.97
5.0	33.57	0.66	5.00	0.09	13.70
6.0	30.86	0.60	6.00	0.11	13.43
7.0	27.98	0.53	7.00	0.12	13.17
8.0	24.94	0.47	8.00	0.14	12.90
9.0	21.74	0.40	9.00	0.16	12.63
10.0	18.40	0.33	10.00	0.18	12.36
11.0	14.92	0.27	11.00	0.19	12.09
12.0	11.32	0.20	12.00	0.21	11.81
13.0	7.63	0.13	13.00	0.23	11.54
14.0	3.87	0.07	14.00	0.25	11.26
15.0	0.08	0.00	15.00	0.27	10.98
16.0	0.00	0.00	16.00	0.29	10.69
17.0	0.00	0.00	17.00	0.31	10.41
18.0	0.00	0.00	18.00	0.32	10.12
19.0	0.00	0.00	19.00	0.34	9.82
20.0	0.00	0.00	20.00	0.36	9.53
21.0	0.00	0.00	21.00	0.38	9.23
22.0	0.00	0.00	22.00	0.40	8.92
23.0	0.00	0.00	23.00	0.42	8.61
24.0	0.00	0.00	24.00	0.45	8.30
25.0	0.00	0.00	25.00	0.47	7.98
26.0	0.00	0.00	26.00	0.49	7.66
27.0	0.00	0.00	27.00	0.51	7.33
28.0	0.00	0.00	28.00	0.53	6.99
29.0	0.00	0.00	29.00	0.55	6.65
30.0	0.00	0.00	30.00	0.58	6.30
31.0	0.00	0.00	31.00	0.60	5.95
32.0	0.00	0.00	32.00	0.62	5.59
33.0	0.00	0.00	33.00	0.65	5.22
34.0	0.00	0.00	34.00	0.67	4.84
35.0	0.00	0.00	35.00	0.70	4.45
36.0	0.00	0.00	36.00	0.73	4.05
37.0	0.00	0.00	37.00	0.75	3.64
38.0	0.00	0.00	38.00	0.78	3.23
39.0	0.00	0.00	39.00	0.81	2.80
40.0	0.00	0.00	40.00	0.84	2.35
41.0	0.00	0.00	41.00	0.87	1.90
42.0	0.00	0.00	42.00	0.90	1.43
43.0	0.00	0.00	43.00	0.93	0.94
44.0	0.00	0.00	44.00	0.97	0.44
45.0	0.00	0.00	45.00	1.00	0.00
46.0	0.00	0.00	46.00	1.04	0.00
47.0	0.00	0.00	47.00	1.07	0.00
48.0	0.00	0.00	48.00	1.11	0.00
49.0	0.00	0.00	49.00	1.15	0.00
50.0	0.00	0.00	50.00	1.19	0.00

Appendix 2.2

Calculation of Rock Bolt

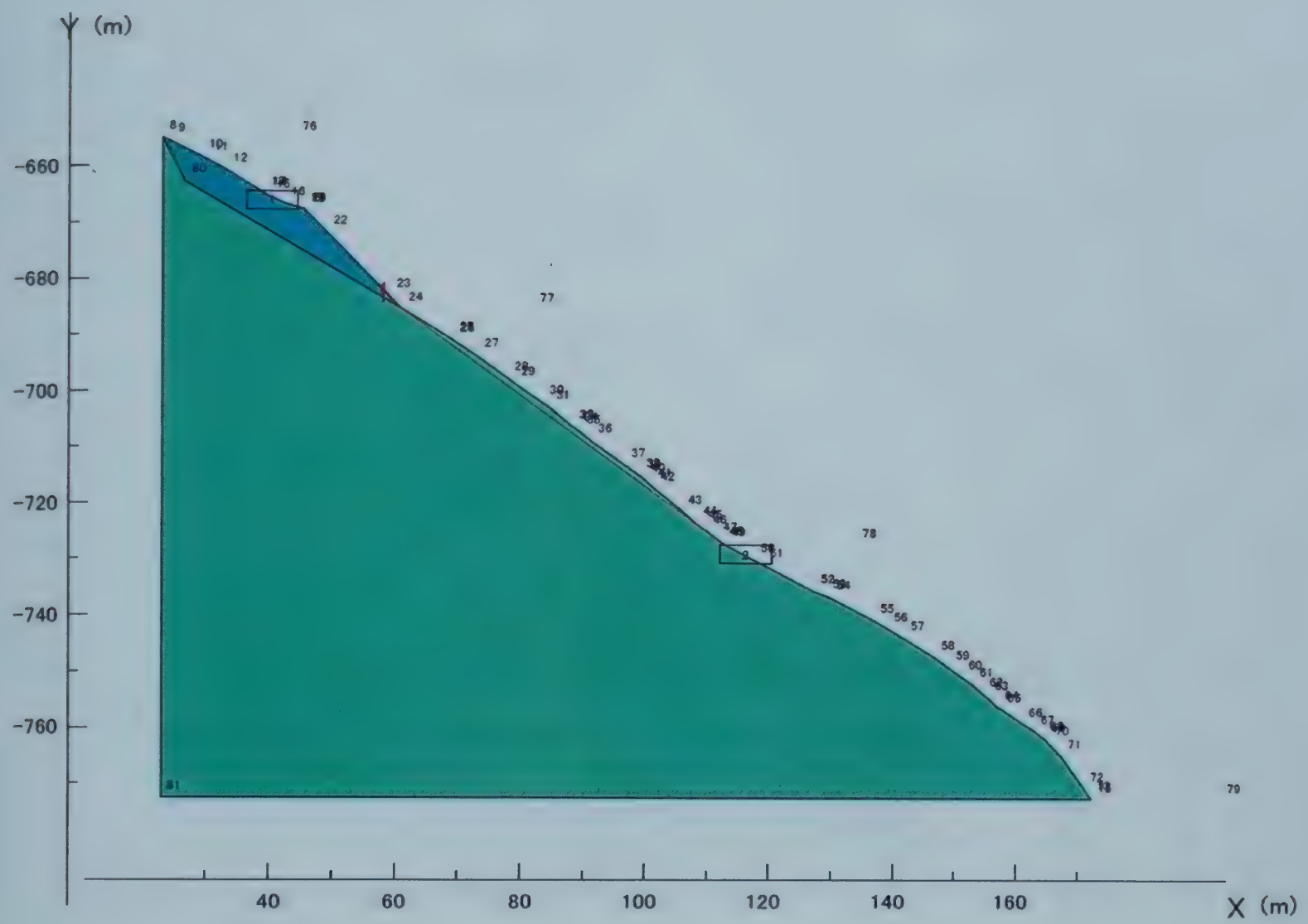
Calculation Result of Rock Bolt

Slide No.3

Calculation Result of Rock Bolt

Slide No.3 - Upper Zone 2)-2

Surface with Maximum Prevention Force (Upper)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-3 Upper(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor					
Plan safety factor of reinforcement slope	Fs	1.20			
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345	
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)	
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	480	(kN/m2)	τp1	480 (kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0			
Reduction coefficient for tensile force	λ	0.7			
slope protection reduction coefficient	μ	0.7			

Condition of slope					
Internal friction angle of soil	φ	40	°	Cohesion	kN/m2
Resistance of landslide	ΣS	564.20	(kN/m)		
Sliding force	ΣT	542.50	(kN/m)	Safety factor	1.040
Necessary deterrent force	PR	86.8	(kN/m)	PR=Fs*ΣT-ΣS	

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm) consideration of the corrosion allowance (nominal diameter−1.0mm)
		24.4	(mm)	consideration of the corrosion allowance (1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	
Horizontal spacing of reinforcement material	SH	6.0	(m)	
Vertical spacing of reinforcement material	SV	4.0	(m)	
Number of steps of reinforcement material	n	10	(Step)	
Allowable bond stress of reinforcement material	ta	75.398	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	75.398	(kN/m)	tpa=(τp*π*D)/Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.205** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	564.2	
Sliding force	ΣT	(kN/m)	542.5	
Resistance power with reinforcemer	Sr	(kN/m)	89.44	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	0.87	1.43	2.30			90.00	218.91	107.74	93.52	93.52	65.46	10.91	0.00	9.15
(2)	0.86	1.44	2.30			90.00	216.39	108.50	93.52	93.52	65.46	10.91	0.00	9.15
(3)	0.87	1.44	2.30			90.00	217.40	108.20	93.52	93.52	65.46	10.91	0.00	9.15
(4)	0.88	1.42	2.30			90.00	220.41	107.29	93.52	93.52	65.46	10.91	0.00	9.15
(5)	1.04	1.26	2.30			90.00	260.63	95.23	93.52	93.52	65.46	10.91	0.00	9.15
(6)	1.26	1.04	2.30			90.00	316.17	78.56	93.52	78.56	54.99	9.17	0.00	7.69
(7)	1.14	1.16	2.30			90.00	287.02	87.31	93.52	87.31	61.12	10.19	0.00	8.55
(8)	1.03	1.27	2.30			90.00	259.37	95.60	93.52	93.52	65.46	10.91	0.00	9.15
(9)	1.06	1.24	2.30			90.00	266.41	93.49	93.52	93.49	65.44	10.91	0.00	9.15
(10)	1.01	1.29	2.30			90.00	254.59	97.04	93.52	93.52	65.46	10.91	0.00	9.15
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													0.00	89.44

Calculating formula

T1pa : Allowable tensile resistance force of reinforcement material subjected to the moving mass — T1pa=(1/1-μ)*L1*ta

T2pa : Allowable tensile resistance force of reinforcement material subjected to the stable ground — T2pa=L2*ta

Tsa : Allowable tensile stress of reinforcement material — Tsa=σsa*As

Tpa : Allowable tensile bearing strength of reinforcement material — Tpa=min (T1pa, T2pa, Tsa)

Td : Design tensile force (1piece per unit) — Td=λ*Tpa

Tm : Design tensile force (1m width per unit) — Tm=Td/SH

Sh : Detention force — Sh=Tm*cosβ

SS : Striction force — Ss=Tm*sinβ*tanφ

Att) Aa stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 10.9 (kN/No.)

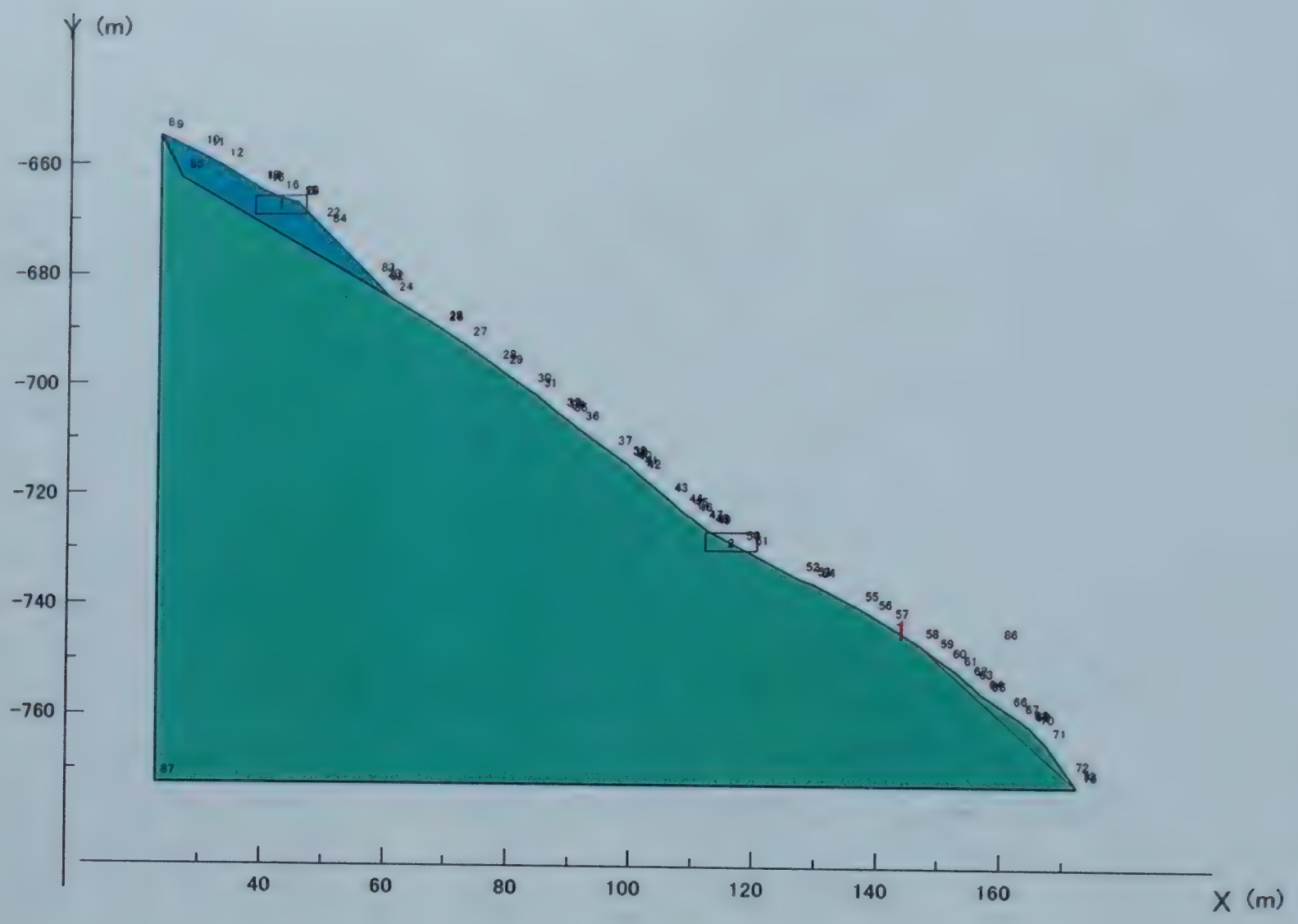
Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.83 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.49 (m)

Calculation Result of Rock Bolt

Slide No.3 - Lower Zone 2)-4

Surface with Maximum Prevention Force (Lower)



Calculation sheet of Rock bolt work

(Site Name) NH10

<Examination Section> Slide-3 Lower(*Topo)

[Cut Earth Reinforcement Method, Design and Construction Procedures]

Allowable stress and limit frictional resistance safety factor						
Plan safety factor of reinforcement slope	Fs	1.20				
Allowable tensile stress of reinfocement material	σsa	200,000	(kN/m2)	SD345		
Allowable bond stress of reinforcement and injection material	τc	1,600	(kN/m2)	σck=24(N/mm2)		
Shaft friction resistence between ground and grout(Tp1,Tp2)	τp2	480	(kN/m2)	τp1	480	(kN/m2)
Factor of safety for shaft friction resistance	Fsa	2.0				
Reduction coefficient for tensile force	λ	0.7				
slope protection reduction coefficient	μ	0.7				

Condition of slope						
Internal friction angle of soil	φ	40	°	Cohesion	2.35	kN/m2
Resistance of landslide	ΣS	542.70	(kN/m)			
Sliding force	ΣT	542.70	(kN/m)	Safety factor	1.000	
Necessary deterrent force	PR	108.5	(kN/m)	PR=Fs*ΣT-ΣS		

Condition of reinforcing material				
Diameter of boring	D	100	(mm)	
Diameter of reinforcement bar	d	25	: nominal diameter	25.4 (mm)
		24.4	(mm)	consideration of the corrosion allowance (nominal diameter−1.0mm)
Sectional area of reinforcement material	As	4.676	(cm2)	consideration of the corrosion allowance (1.0mm)
Horizontal spacing of reinforcement material	SH	4.0	(m)	
Vertical spacing of reinforcement material	SV	4.0	(m)	
Number of steps of reinforcement material	n	8	(Step)	
Allowable bond stress of reinforcement material	ta	75.398	(kN/m)	ta= min {tpa, tca}
Allowable adherence between ground and grout	tpa	75.398	(kN/m)	tpa=(τp*π*D)/Fsa
Allowable between reinforcement material and grout	tca	122.648	(kN/m)	tca=τc*π*d

Weight and sectional area of deformed steel bar

Alias	Weight (kg/m)	Nominal diameter (mm)	Nominal cross-sectional area(cm2)
D19	2.250	19.1	2.865
D22	3.040	22.2	3.871
D25	3.980	25.4	5.067
D29	5.040	28.6	6.424
D32	6.230	31.8	7.942

Indication of type of slope protection and reduction coefficient

Slope protection type	μ	Remark
Planting work	0	The reinforcement head is not fixed
Shotcrete work	0.2～0.6	The heads are connected in the head plate etc.
Crib work	0.7～1.0	The heads are united enough
Retaining wall	1.0	Consecutive board type slope

Calculation Result

Item			Calculation Result	
Safety factor after reinforcement	Fs		**1.212** > 1.20	(ΣS+Sr)/ΣT
Resistance of landslide	ΣS	(kN/m)	542.7	
Sliding force	ΣT	(kN/m)	542.7	
Resistance power with reinforcemer	Sr	(kN/m)	114.93	(Sh+Ss)

Conditions of performance						Design of tensile force of the each reinforcement								
Reinforcement material №	Fixed length of movement L1(m)	Fixed length of stable L2(m)	Drilling length L (m)	Angle of the slip surface θ(°)	Installation angle of reinforcement material α(°)	β (°)	T1pa (kN)	T2pa (kN)	Tsa (kN)	Tpa (kN/No.)	Td (kN/No.)	Tm (kN/m)	Sh (kN/m)	Ss (kN/m)
(1)	0.96	2.34	3.30			77.47	241.53	176.36	93.52	93.52	65.46	16.37	3.55	13.40
(2)	1.85	1.45	3.30			77.47	464.96	109.33	93.52	93.52	65.46	16.37	3.55	13.40
(3)	2.29	1.01	3.30			85.25	575.79	76.08	93.52	76.08	53.26	13.32	1.10	11.13
(4)	1.94	1.36	3.30			98.36	488.08	102.39	93.52	93.52	65.46	16.37	0.00	13.59
(5)	1.35	1.95	3.30			98.33	340.30	146.72	93.52	93.52	65.46	16.37	0.00	13.59
(6)	1.00	2.30	3.30			88.19	250.57	173.64	93.52	93.52	65.46	16.37	0.52	13.73
(7)	0.90	2.40	3.30			90.97	226.95	180.73	93.52	93.52	65.46	16.37	0.00	13.73
(8)	0.48	2.82	3.30			96.56	119.88	212.85	93.52	93.52	65.46	16.37	0.00	13.64
(9)														
(10)														
(11)														
(12)														
(13)														
(14)														
(15)														
(16)														
(17)														
(18)														
Sum													8.72	106.21

Calculating formula

T1pa :	Allowable tensile resistance force of reinforcement material subjected to the moving mass	T1pa=(1/1-μ)*L1*ta	Td :	Design tensile force (1piece per unit)	Td=λ*Tpa
T2pa :	Allowable tensile resistance force of reinforcement material subjected to the stable ground	T2pa=L2*ta	Tm :	Design tensile force (1m width per unit)	Tm=Td/SH
Tsa :	Allowable tensile stress of reinforcement material	Tsa=σsa*As	Sh :	Detention force	Sh=Tm*cosβ
Tpa	Allowable tensile bearing strength of reinforcement material	Tpa=min (T1pa, T2pa, Tsa)	SS	Striction force	Ss=Tm*sinβ*tanφ

Adj) As stick of design tensile force Td=PR/(cosβ+sinβ*tanφ)×Dh/n/λ= 16.4 (kN/No.)

Necessary bond length between bedrock and injection material Fsa*Td/π/D/τp= 0.70 (m)

Bond length between injection material and lock bolts Td/π/d/τc= 0.42 (m)

Calculation of Crib Works

Calculation of Crib Works

Slide 3

Calculation of Crib Works

Slide 3 - Lower 2)-2

Slide-3 Lower

Design load　Td=　16.4 kN／No.　(Td-value of rock bolt)

Design condition

Span of the transversal frame	Lx=	2.00	m
Span of the longitudinal frame	Ly=	2.00	m
material	Dh=	2.00	m
Longitudinal space of the reinforcing material	Dv=	2.00	m
Width of the frame	b=	300	mm
Hight of the frame	h=	300	mm
Head space of the frame	d=	235	mm
Design strength of the mortar	σck=	18	N/mm2
Allowable compressive stress	σca=	6	N/mm2
Allowable shear stress	τca=	0.40	N/mm2
Allowable adhesive stress of reinforcing bar and mortar	τoa=	1.4	N/mm2
Allowable tensile strength of reintocing bar(SD295)	σsa=	180	N/mm2

Section of the frame	Standard head space
200	153
300	235
400	315
500	410

Main reinforcing bar　D13 ×上下各　2 本

126.7

Calculation result

Uniformly-distributed load	w=	4.43	kN/m
Maximum bending moment	M=	2.22	kN･m
Maximum shear strength	S=	4.43	kN
cross-sectional area of main reinforcing bar	As=	253.4	mm2
	p=	0.00359	kN/m
	n=	15	
	k=	0.27872	
	j=	0.907	
	1/Lc	7.91	
	1/Ls	307.11	

◎Compressive stress of the mortar	σc=	1.1	N/mm2	O K	6
◎Tensile stress of reinforcing bar	σs=	41.2	N/mm2	O K	180
◎Shear stress of mortar	τc=	0.07	N/mm2	O K	0.40
◎Adhesive stress of mortar and reinforcing bar	τo=	0.13	N/mm2	O K	1.4

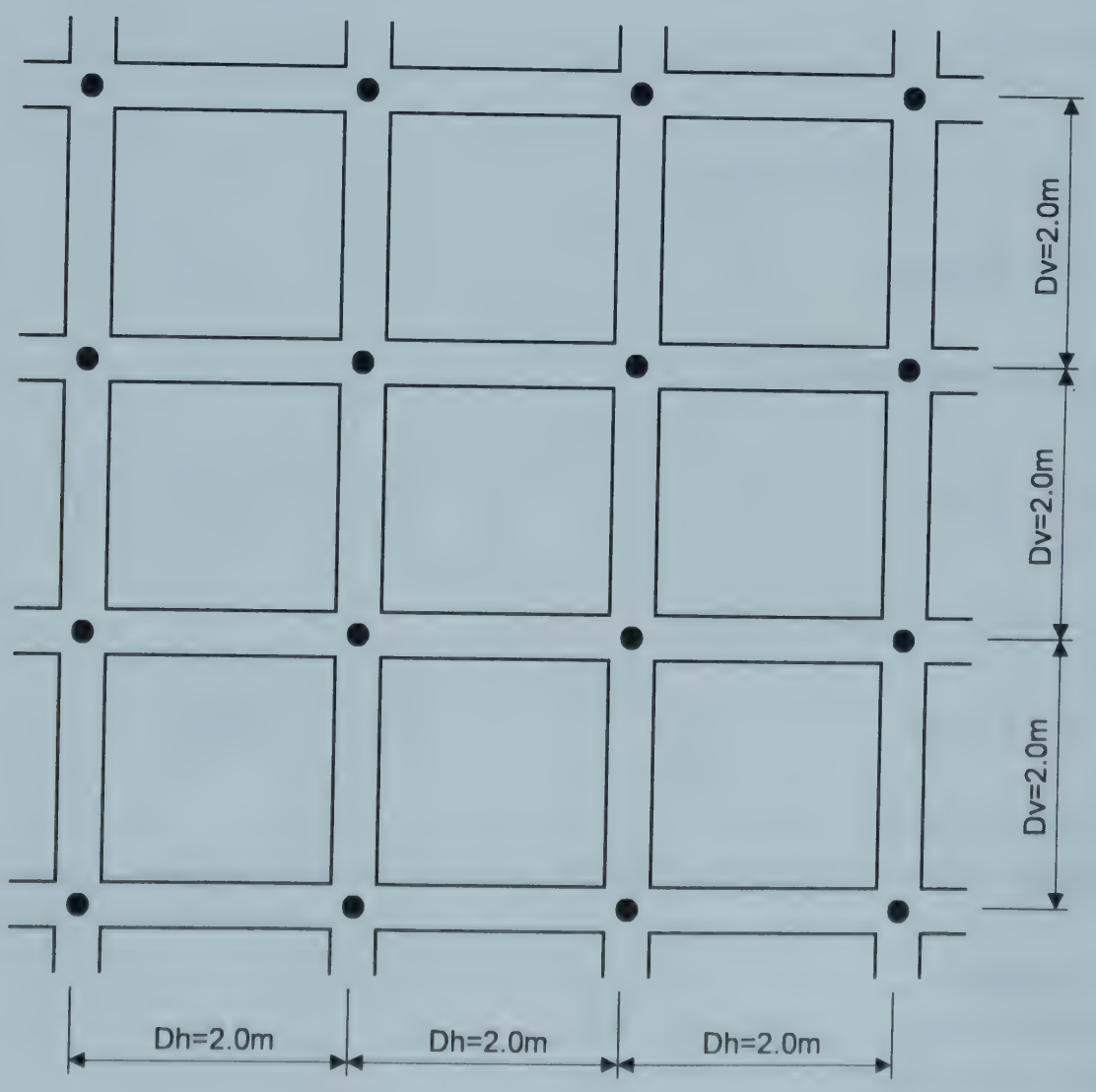
Weight, sectional area, and peripheral length of strange appearance steel b

Alias	Unit mass	Nominal diamete	Nominal cross-	Nominal perimeter
D10	0.560	9.53	71.33	30
D13	0.995	12.7	126.7	40
D16	1.56	15.9	198.6	50
D19	2.25	19.1	286.5	60
D22	3.04	22.2	387.1	70
D25	3.98	25.4	506.7	80
D29	5.04	28.6	642.4	90
D32	6.23	31.8	794.2	100

<Design calculation of crib work >

① Design Condition

Design load (Maxmum of Td)	Td	=	16.4 kN／No.
Span of the transversal frame	Lx	=	2.00 m
Span of the longitudinal frame	Ly	=	2.00 m
Transverse space of the reinforcing material	Dh	=	2.00 m
Longitudinal space of the reinforcing material	Dv	=	2.00 m
Width of the frame	b	=	300 mm
Hight of the frame	h	=	300 mm
Head space of the frame	d	=	235 mm
Design strength of the mortar	σ_{ck}	=	18 N/mm^2
Allowable compressive stress	σ_{ca}	=	6 N/mm^2
Allowable shear stress	τ_{ca}	=	0.40 N/mm^2
Allowable adhesive stress of reinforcing bar and mor	τ_{oa}	=	1.4 N/mm^2
Allowable tensile strength of reinfocing bar(SD2S	σ_{sa}	=	180 N/mm^2



Layout drawing of reference chart 1(crib work)

②Decision of kind of bea

(a)The subgrade reaction on the bedrock is not clear.

(b) The rigidity of the length and breadth frame is equal.

(c) The length and breadth span is equal (not twice or more).

From the above-mentioned and (a)

It is safe that the soil reaction acts on the frame as a uniformly-distributed load

Moreover, examining it from (b) and (c) as two direction beam is suitable.

③Bending moment and shear pow

·Calculation of uniformly-distributed load

$$w = \frac{Td}{Lx+Ly-b} = \frac{16.4}{2.00 + 2.00 - 0.30}$$

Lx=Ly= 2.00 m

b= 0.30 m

$$= 4.43 \text{ kN/m}$$

·Maximum bending moment

$$M = \frac{1}{8} w \cdot L^2$$

L=Lx=Ly= 2.00 m

$$= \frac{1}{8} \times 4.43 \times 2.00^2$$

$$= 2.22 \text{ kN·m}$$

·The maximum shear stress

$$S = \frac{1}{2} w \cdot L$$

$$= \frac{1}{2} \times 4.43 \times 2.00$$

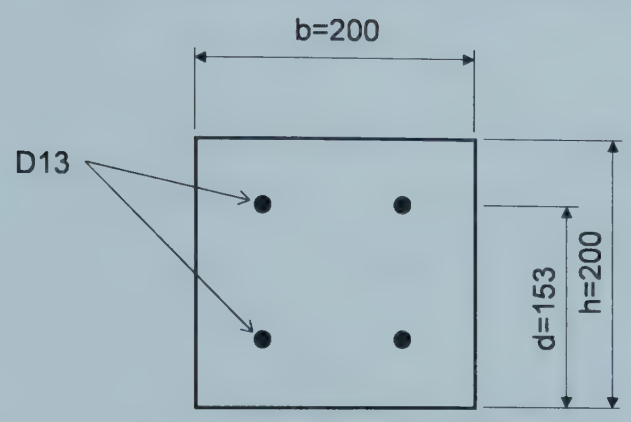
$$= 4.43 \text{ kN}$$

④ Check of stress degree

The stress degree is checked as a single reinforced concrete rectangular section.

·Setting of section

Width of the frame	b	=	300 mm
Head space of the frame	d	=	235 mm
Reinforcing bar	D13 ×	Upper and low	2 pieces



Reference chart 2 section

$$p = \frac{As}{b \cdot d} = \frac{253.4}{300 \times 235}$$

$$= 0.00359$$

$$n = \#$$

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$= \sqrt{2 \times \# \times 0.00359 + (\# \times 0.00359)^2} - \# \times 0.00359$$

$$= 0.27872$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.27872}{3}$$

$$= 0.91$$

$$\frac{1}{Lc} = \frac{2}{k \cdot j} = \frac{2}{0.27872 \times 0.91}$$

$$= 7.91$$

$$\frac{1}{Ls} = \frac{1}{p \cdot j} = \frac{1}{0.00359 \times 0.91}$$

$$= 307.11$$

・Compressive stress of the mortar

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 7.91$$

$$= 1.1 \ N/mm^2 \quad < \quad \sigma_{ca} = 6 \ N/mm^2 \qquad OK$$

・Tensile stress of reinforcing bar

$$\sigma s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls}$$

$$= \frac{2.22 \times \#^{6}}{300 \times 235^{2}} \times 307.11$$

$$= 41.2\ N/mm^2 < \sigma_{sa} = 180\ N/mm^2 \quad OK$$

・Shear stress of mortar

$$\tau c = \frac{S}{b \cdot j \cdot d}$$

$$= \frac{4.43 \times \#^{3}}{300 \times 0.907 \times 235}$$

$$= 0.07\ N/mm^2 < \tau_{ca} = 0.40\ N/mm^2 \quad OK$$

・Adhesive stress of mortar and reinforcing bar

$$\tau c = \frac{S}{U \cdot j \cdot d}$$

U:entire circumference length of reinforcing bar $= \# \text{ mm} \times 2$ Pieces

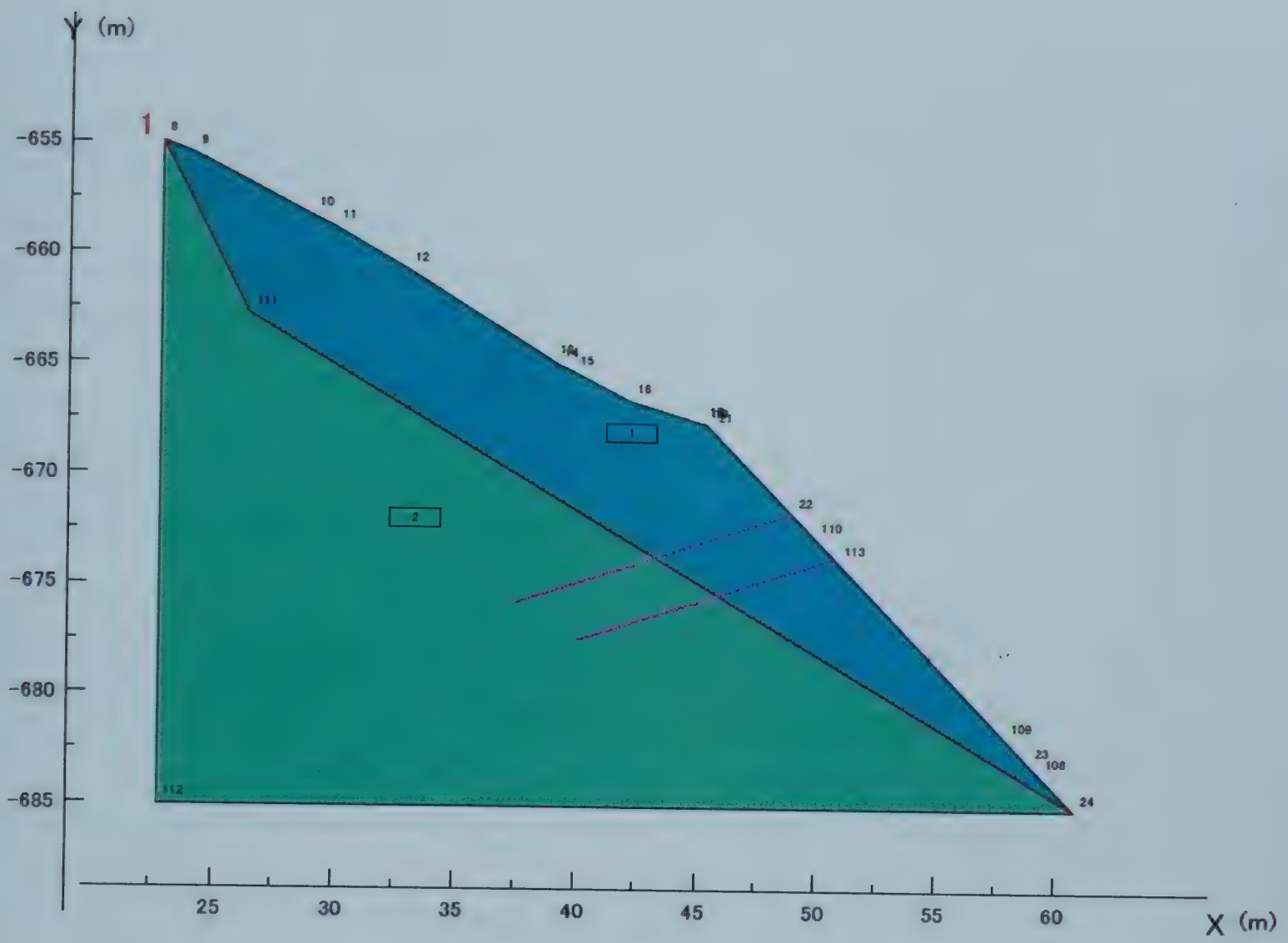
$$= \frac{4.43 \times \#^{3}}{(\# \times 2) \times 0.91 \times 235}$$

$$= 0.26\ N/mm^2 < \tau_{oa} = 1.4\ N/mm^2 \quad OK$$

Appendix 2.4

Calculation of Ground Anchor

Ground Anchor Design


(m)
-655
-660
-665
-670
-675
-680
-685
1
8
9
10
11
12
111
15
16
21
1
2
22
110
113
109
23
108
24
112
25
30
35
40
45
50
55
60
X (m)

Ground Anchorage Design Calculation Report

Area	National Highway 10 - Slide 3
Traverse line	
Remarks	

Item		Symbol	Unit	Numerical value
Required prevention force		Pr	kN/m	385.00
Sliding surface	Average angle	θ	°	33.00
	$\tan\phi$	-	-	0.65 (ϕ =33.000°)
Horizontal interval		a	m	2.50
Number of constructed steps		m	step	2
Angle of inclination of anchorage		α	°	20.00
Effect of anchorage		Stressing + Anchoring effect		
Materials and type of anchorage		PC steel strand(Permanent ground anchorage) ϕ 12.7		
Rank of anchorage		Rank A (Normally)		
Allowable adhesive stress tendon and grout		τ b	N/mm^2	0.80
Skin friction of fixed anchor		τ	N/mm^2	0.60
Design safety factor		Fs	-	2.50

Calculation results

Angle of inclination of anchorage α (°)	Working load of anchorage Td(kN/pc.)	Tendon standard	Apparent circumference U(mm)	Borehole diameter dA(mm)	Fixed anchor length (m)		
					las	la	La
20.0	430.0	ϕ 12.7×4	130.6	115	4.12	4.96	5.00

1. Ground Anchorage Design Calculation

1.1. Design Requirements

1.1.1. Sliding Surface

(1) Required prevention force	Pr = 385.00 (kN)
(2) Sliding surface gradient	θ = 33.00 (°)

1.1.2. Layout and Functions of Ground Anchorage

(1) Horizontal interval of anchorage	a = 2.50 (m)
(2) Number of constructed steps	m = 2 (step)
(3) Angle of inclination of anchorage	α = 20.0 (°)
(4) Angle formed between anchorage and sliding surface	$\beta = \alpha + \theta$ =53 (°)
(5) Inhibiting function of anchorage	Stressing + Anchoring effect
(6) Internal friction angle of sliding surface	$\tan\phi$ = 0.6494 (ϕ =33.000°)

1.1.3. Fixed Anchor

(1) Applicable standards	Japanese Geotechnical Society
(2) Supporting system of anchorage	Friction draft type
(3) Materials and type of anchorage	PC steel stand (Permanent ground anchorage) ϕ 12.7
	Rank A (Normally)
(4) Allowable adhesive stress of tendon and grout	τ b= 0.80 (kN/mm^2)

Allowable adhesive stress

(N/mm^2)

			Standard Strength of grout design			
			18	24	30	40 or more
Type of tension member	Rank B	PC steel Strand PC multi-Strand	1.00	1.20	1.35	1.50
		Deformed PC steel bar	1.40	1.60	1.80	2.00
	Rank A	PC steel strand PC multi-strand		0.80	0.90	1.00
		Deformed PC steel bar		1.60	1.80	2.00

The allowable adhesive stress for temporary construction is 1.5m times higher than the permanent ground anchorage for the PC steel and PC multi-strand, and the same as the permanent ground anchorage
for the deformed PC steel bar.

Source: Design and Construction Standards for Ground Anchorage
(Issued by Japanese Geotechnical Society on May 31 2012)

(5) Skin fiction of the fixed anchor τ = 0.60 (N/mm^2)

Skin friction of ground anchorage

Type of ground			Friction resistance (N/mm^2)
Bedrock	Hard rock		1.50 ～ 2.50
	Soft rock		1.00 ～ 1.50
	Weathered rock		0.60 ～ 1.00
	Hardpan		0.60 ～ 1.20
Sand and gravel	N-value	10	0.10 ～ 0.20
		20	0.17 ～ 0.25
		30	0.25 ～ 0.35
		40	0.35 ～ 0.45
		50	0.45 ～ 0.70
Sand	N-value	10	0.10 ～ 0.14
		20	0.18 ～ 0.22
		30	0.23 ～ 0.27
		40	0.29 ～ 0.35
		50	0.30 ～ 0.40
Cohesive soil			1.0c (c:cohesion)

Source: Design and Construction Standards for Ground Anchorage (Issued by Japanese Geotechnical Society on May 31 2012)

(6) Design safety factor Fs= 2.5

Safety factor for ultimate pull-out capacity

		Safety factor
Rank B		1.5
Rank A	(Normally)	2.5
	(At earthquake)	1.5 ～ 2.0

Source: Design and Construction Standards for Ground Anchorage (Issued by Japanese Geotechnical Society on May 31 2012)

(7) Borehole diameter (fixed anchor diameter) dA= 115 (mm)

1.2. Calculation of Working Load of Anchorage

1.2.1. Calculation of Required Anchorage Force

The following shows the relationship between the planned safety factor(PFs) and required prevention force(Pr) of the slope after executing a countermeasure.

$$PFs=\frac{[\text{Resistance to slide}]+Pr}{[\text{Sliding force}]}$$

$$Pr=PFs\cdot[\text{Sliding force}]-[\text{Resistance to slide}]$$

Suppose the required anchorage force is assumed to be Po, the formula, which regards both the stressing force to press against the sliding surface by the anchorage and the anchoring force to pull up along the sliding surface as "resistance to slide," is as follows.

$$PFs=\frac{[\text{Resistance to slide}]+Po\cdot\sin\beta\cdot\tan\phi+Po\cdot\cos\beta}{[\text{Sliding force}]}$$

$$PFs=\frac{PFs\cdot[\text{Sliding force}]-[\text{Resistance to slide}]}{\cos\beta+\sin\beta\cdot\tan\phi}$$

Accordingly, the required anchorage force is calculated as follows in case of the stressing + anchoring effect.

$$Po=\frac{Pr}{\cos\beta+\sin\beta\cdot\tan\phi}$$

$$=\frac{385}{\cos(53)+\sin(53)\cdot 0.6494}=343.61\quad(kN/m)$$

1.2.2. Calculation of Working Load of Anchorage

The working load of anchorage (Td) is calculated as follows.

$$PFs=\frac{Po\cdot a}{m}$$

$$=\frac{343.61\times 2.50}{2}\qquad=429.51\quad(kN/pc.)$$

Based on the above, the working load of anchorage (Td) per piece is 430 kN/pc.

1.3. Determination of Tendon Standard

Materials and type of anchorage : PC steel standard (Permanent ground anchorage) ϕ 12.7
Tendon standard : ϕ 12.7×4

In the above-mentioned case, the allowable tension is calculated as follows respectively with respect to tensile strength(Tus) and yield strength (Tys).

0.60·(Tus·N) =	0.60×732.00 =	439.200 (kN)	≧	430 (kN/pc.)	-	OK
0.75·(Tys·N) =	0.75×624.00 =	468.000 (kN)	≧	430 (kN/pc.)	-	OK

N: Number of PC steel standards (4 strands)
Tus: Tensile strength (183.000kN per strand)
Tys: Yield strength (156.000kN per strand)

Based on the above, ϕ 12.7×4 is safe for the working load of anchorage.

1.4. Calculation of Fixed Anchor Length

1.4.1. Calculation Requirements

Allowable adhesive stress of tendon and grout	τb =	0.8 (N/mm^2)
Skin friction of fixed anchor	τ =	0.6 (N/mm^2)
Design safety factor	Fs =	2.5
Tendon circumference	U =	130.6 (mm)
Borehole diameter(fixed anchor diameter)	dA =	115 (mm)

1.4.2. Calculation Results

(1) Tendon restraint length

Tendon restraint length calculated from adhesion between the grout and anchor tendon.

$$las = \frac{Td \cdot 10^3}{U \cdot \tau b}$$

$$= \frac{430 \cdot 10^3}{130.6 \times 0.80} = 4115.6\,(mm) \quad = 4.12\,(m)$$

(2) Calculation of the fixed anchor length

Fixed anchor length calculated from friction between the grout and the ground.

$$la = \frac{Td \cdot 10^3 \times Fs}{\pi \cdot dA \cdot \tau}$$

$$= \frac{430 \times 10^3 \times 2.5}{\pi \times 115 \times 0.60} = 4959.18\,(mm) \quad = 4.96\,(m)$$

(3) Determination of the fixed anchor length

Based on the following ; las =4.12(m) < la = 4.96(m)

The fixed anchor length(La) is La =5.0m

Appendix 2.5

Drawings

(Plan View, Representative Cross sections,

Representative drawing of crib works,

Representative drawing of ground anchor pillow and anchor plate)

Slide-3

Rock fall protection net with rock bolt

The rock fall protection net with rock bolts should retain detached fallen rock at the original position before falling from the escarpment. The dominant size of the fallen rocks is Φ 300 ~ 400 mm with a maximum of Φ 1,000 mm.

Upper horizontal drainage borings:

L=40m, N=8, Interval=5m, Angle of drilling =5degrees

Ground anchor works:

430 kN/pc, 2Steps, Interval=2.5m, Angle of drilling=20degrees
(Upper L=12.5m, N=15, Lower L=11.5m, N=14)
Bearing plate VL=2.00m, HL=2.00m, D=0.5m

Retaining wall:

Gabion L=1.2m, W=1.0m, H=0.5m, 6steps

Upper rock bolt:

10Steps, Interval=6.0m, L=2.3(+0.1)m

Shotcrete:

D=0.1m

Lower horizontal drainage borings:

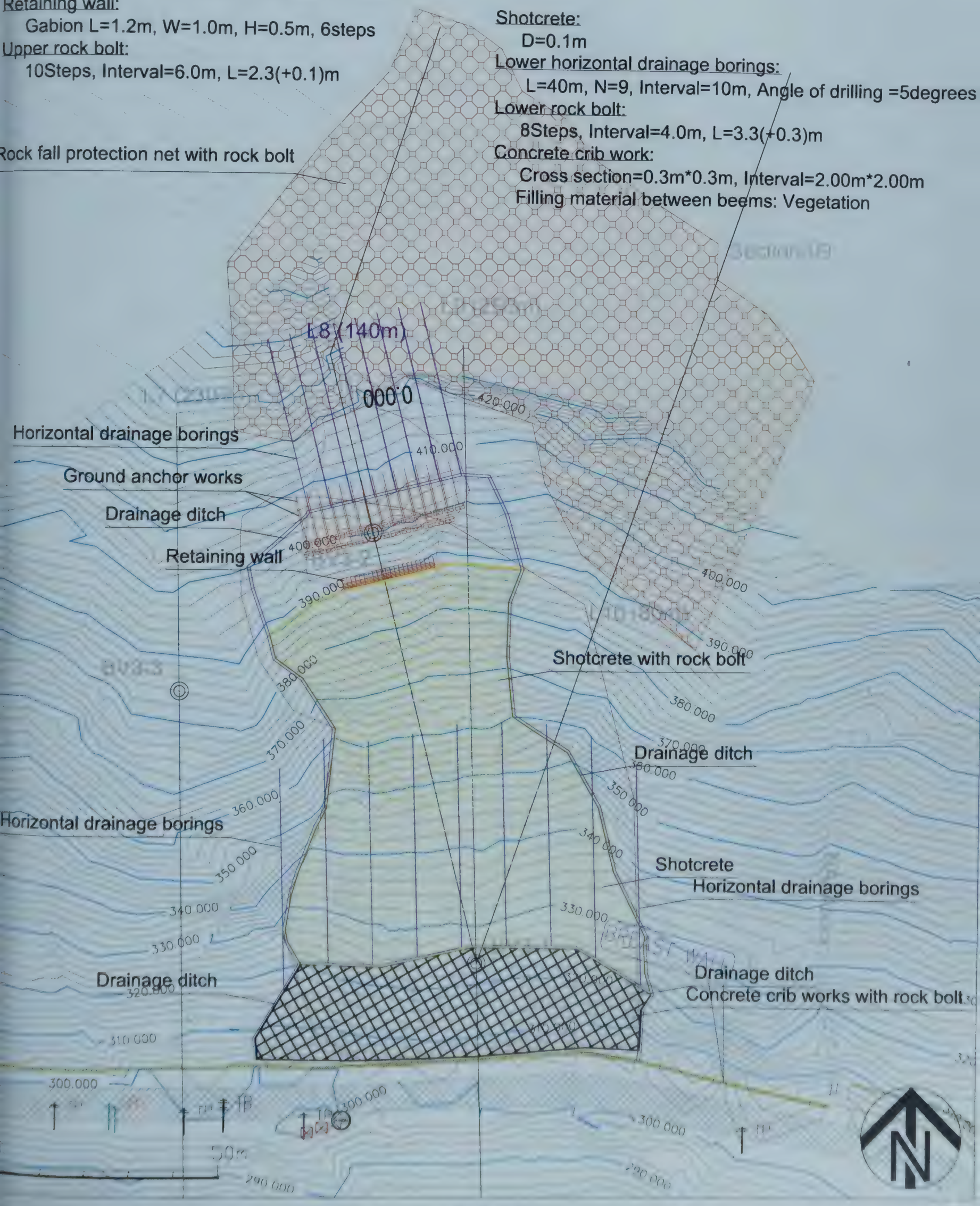
L=40m, N=9, Interval=10m, Angle of drilling =5degrees

Lower rock bolt:

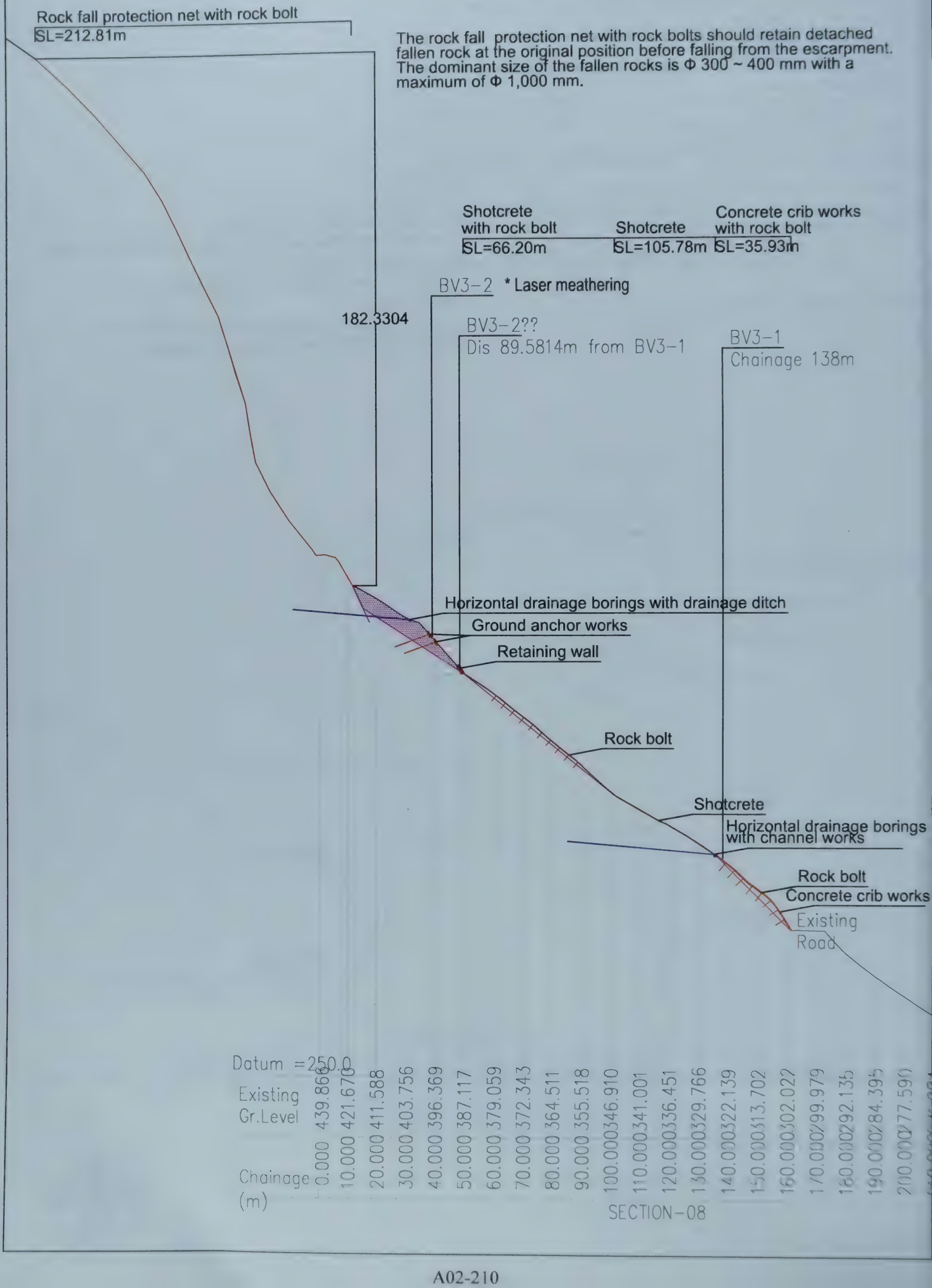
8Steps, Interval=4.0m, L=3.3(+0.3)m

Concrete crib work:

Cross section=0.3m*0.3m, Interval=2.00m*2.00m
Filling material between beems: Vegetation



Slide-3


Rock fall protection net with rock bolt
SL=212.81m
The rock fall protection net with rock bolts should retain detached fallen rock at the original position before falling from the escarpment. The dominant size of the fallen rocks is Φ 300 ~ 400 mm with a maximum of Φ 1,000 mm.
Shotcrete with rock bolt
SL=66.20m
Shotcrete
SL=105.78m
Concrete crib works with rock bolt
SL=35.93m
BV3-2 * Laser meathering
182.3304
BV3-2??
Dis 89.5814m from BV3-1
BV3-1
Chainage 138m
Horizontal drainage borings with drainage ditch
Ground anchor works
Retaining wall
Rock bolt
Shotcrete
Horizontal drainage borings with channel works
Rock bolt
Concrete crib works
Existing Road
Datum =250.0
Existing Gr.Level
Chainage (m)
0.000 439.866
10.000 421.670
20.000 411.588
30.000 403.756
40.000 396.369
50.000 387.117
60.000 379.059
70.000 372.343
80.000 364.511
90.000 355.518
100.000346.910
110.000341.001
120.000336.451
130.000329.766
140.000322.139
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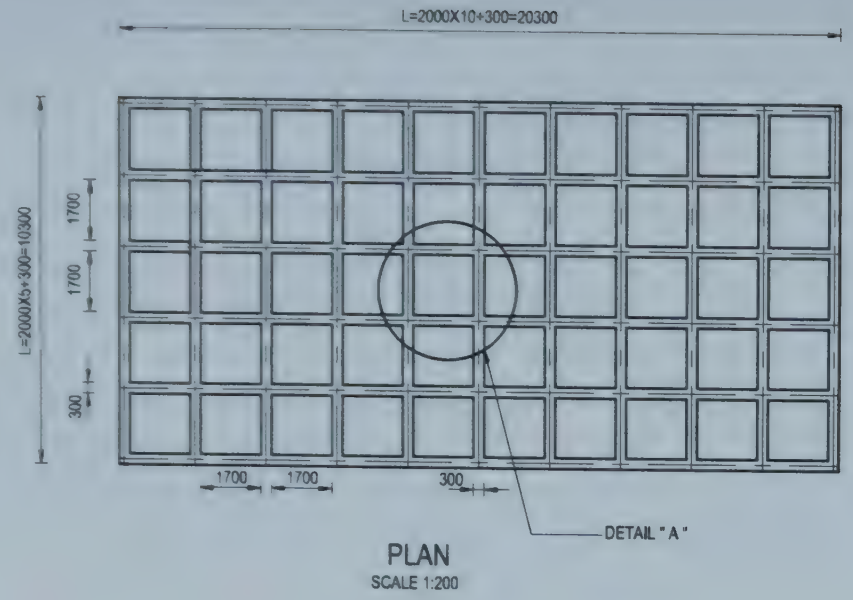
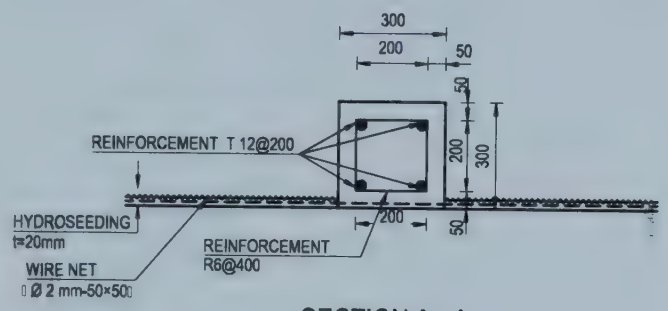
180.000292.135
190.000284.395
200.000277.590
SECTION-08



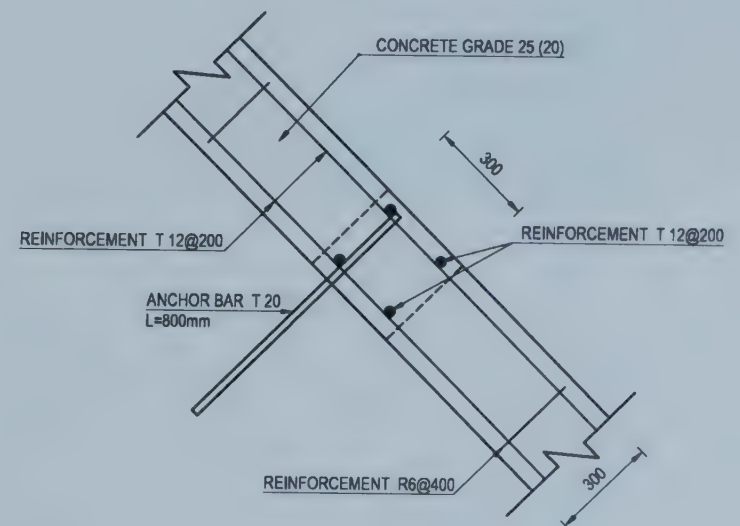
PLAN
SCALE 1:200

QUANTITY CALCULATIONS FOR CRIB WORK AREA OF 210 m² (20.3 m x 10.3 m)

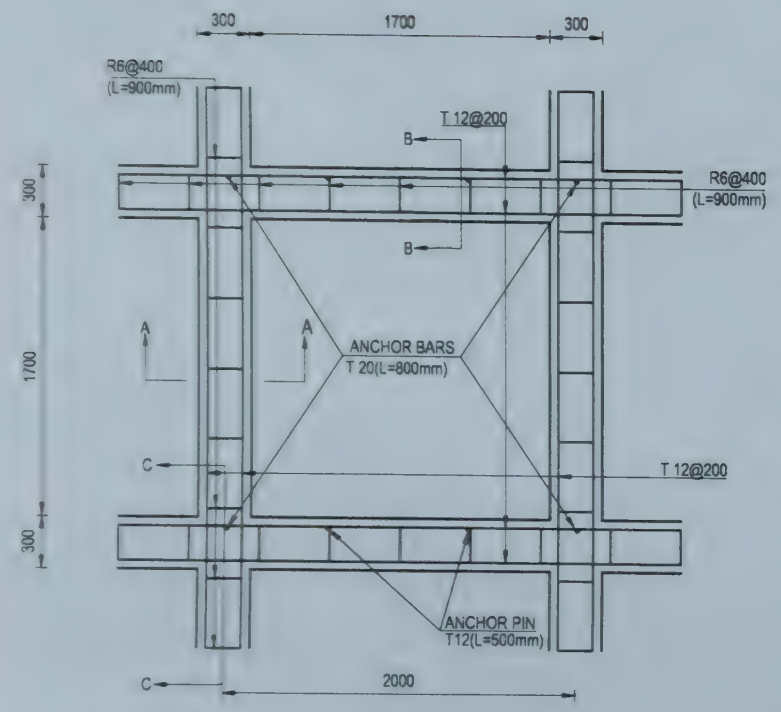
TYPE		UNIT	QUANTITY	CALCULATION FORMULA (Kg)	
FRAME LENGTH		m	215.3	10.3x11+1.7x10x6	
ANCHOR BAR (T20, 800 mm)		nos	66.0	215.30 11x6	66.00
ANCHOR PIN (T12 500mm)		nos	120.0	2x(10x6)	120.00
REINFORCEMENT WEIGHT	T	t	0.84	20.3x6x4x0.888	432.63
				10.3x11x4x0.888	402.44
	R		0.11	0.90x5x55x0.222	54.95
				0.90x5x60x0.222	59.94
				TOTAL	949.96
CONCRETE GRADE 25(20)		m^3	19.4	215.3x0.30x0.30	19.38
AREA WITHIN THE CRIB		m^2	144.5	1.70x1.70x50	144.50



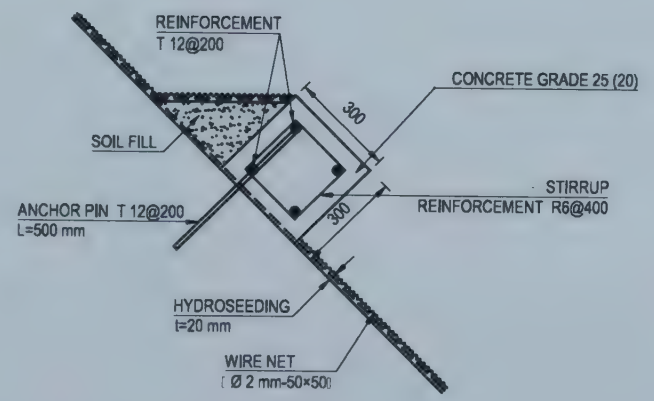
SECTION A - A
SCALE 1:20



SECTION C - C
SCALE 1:20



DETAIL " A "
SCALE 1:40



SECTION B - B
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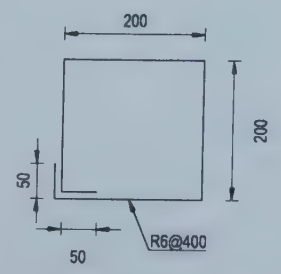

STIRRUP DETAIL
SCALE 1:10

NOTE
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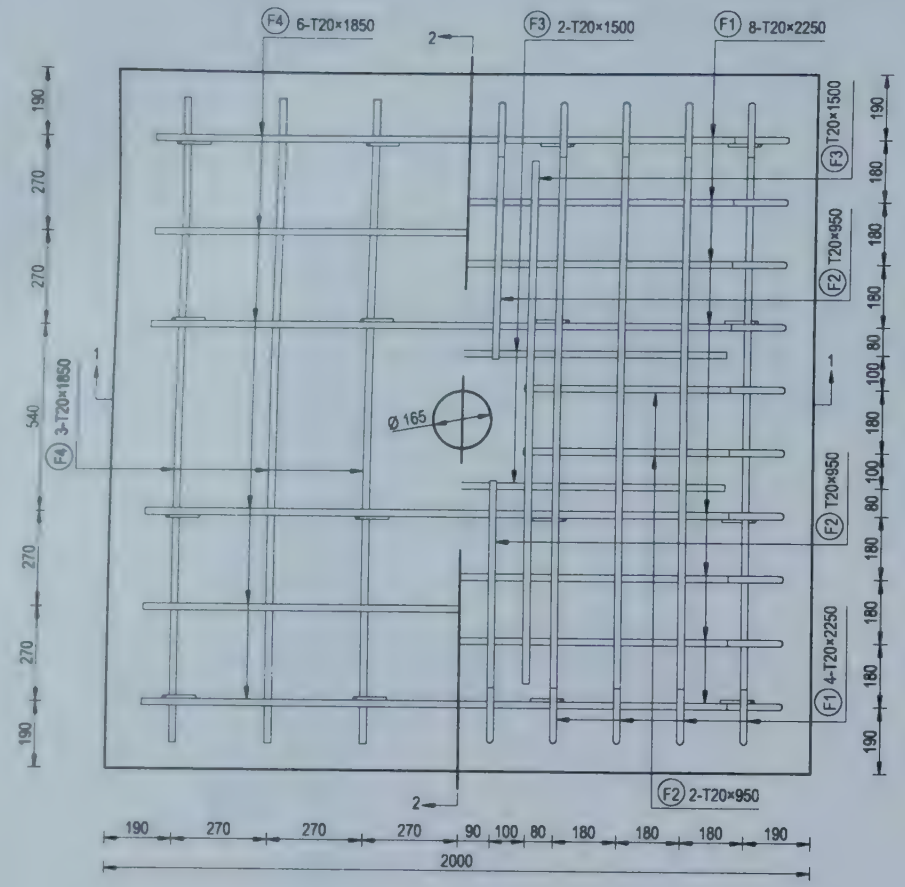
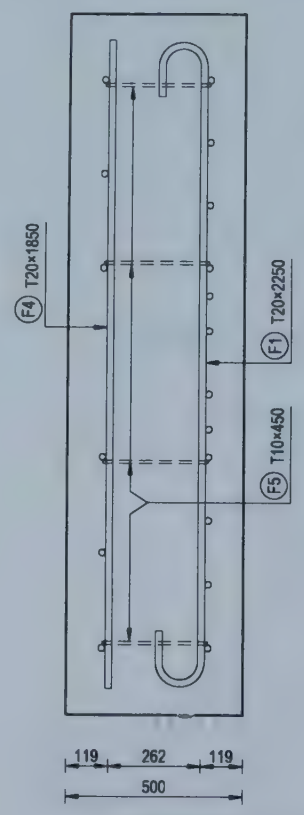
A02-211

Representative drawing of ground anchor pillow and anchor plate



PLAN
SCALE 1:20



SECTION 2-2
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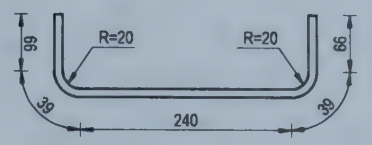
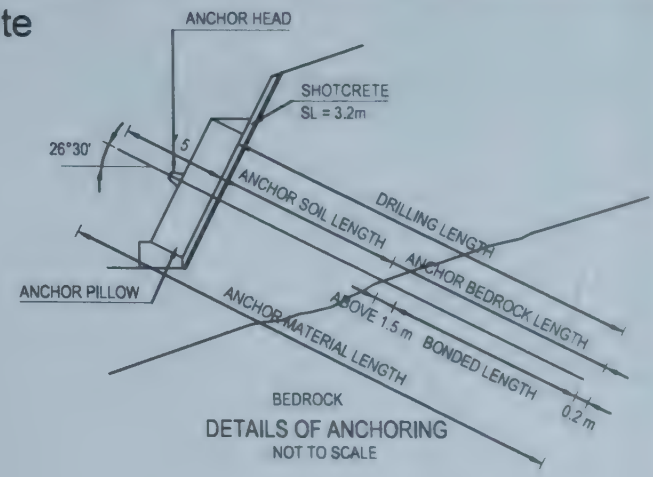
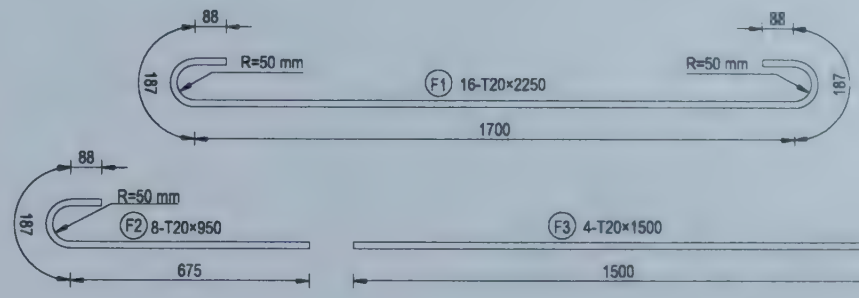


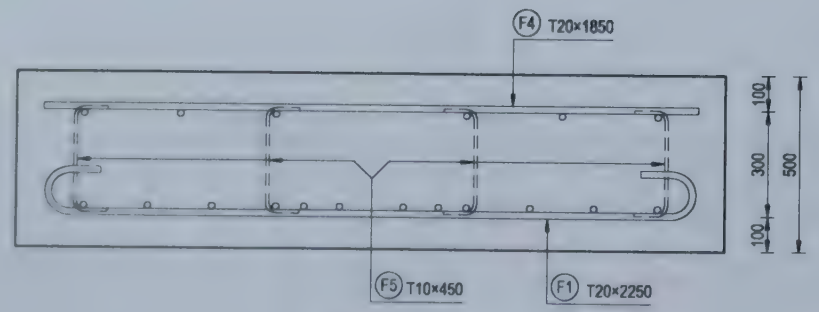
SPACER BAR
SCALE 1:8



DETAILS OF ANCHORING
NOT TO SCALE

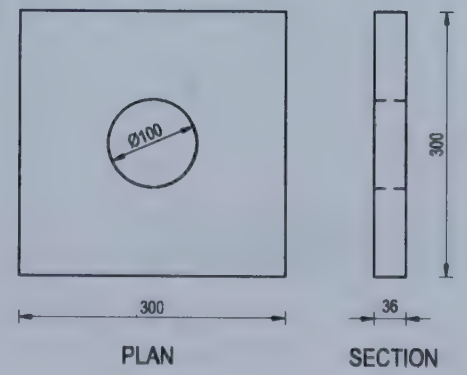


REINFORCEMENT BAR
SCALE 1:20



SECTION 1-1
SCALE 1:20

ANCHOR PILLOW



PLAN SECTION

ANCHOR PLATE
SCALE 1:8

DETAILS OF CONSTRUCTION MATERIALS

1.	STEEL BAR		SD295A	
2.	ANCHOR PLATE		SS 400	
3.	CONCRETE	DESIGN STANDARD STRENGTH	25	kgf / cm
		SLUMP	8	cm
		MAXIMUM COARSE AGGREGATE SIZE	25	mm

WEIGHT OF STEEL

BAR MARK	BAR SIZE (mm)	LENGTH (mm)	NUMBERS (Nos)	UNIT WEIGHT (kg/m)	WEIGHT PER BAR (kg)	WEIGHT kg
F1	T20	2250	16	2.25	5.06	80.96
F2	T20	950	8	2.25	2.14	17.12
F3	T20	1500	4	2.25	3.38	13.52
F4	T20	1850	12	2.25	4.16	49.92
F5	T10	450	16	0.56	0.25	4.00
TOTAL 20mm STEEL BAR WEIGHT						161.52
TOTAL 10mm STEEL BAR WEIGHT						4.00
TOTAL STEEL BAR WEIGHT						165.52
ANCHOR PLATE						25.43

NOTE

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Appendix III

Recommendation on Countermeasure Design: NH54

Recommendations on Countermeasures against Gravity Deformation on NH54 in the State of Assam

October 2021

JICA Experts for Capacity Development Project on Highways in Mountainous Regions

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Appendix7: Image Drawing of Drainage Well

Appendix8: Calculation for Drainage Well on L2

1 Introduction

As a part of Pilot Projects, this report summarizes technical recommendation on countermeasures against gravity deformation at the pilot project site on NH54 in the State of Assam.

At the site, the gravity deformation blocks lying on hill side of NH54 cause the collapse of the cut slopes. Thus slope protection works on the cut slope will be damaged without implementing proper countermeasures to settle / stabilize the gravity deformation blocks.

In this recommendation, the properties of the gravity deformation blocks affecting the cut slope are estimated and then countermeasures to stabilize / settle gravity deformation are examined through slope stability analysis, while leaving remarks on implementation.

We expect this recommendation would help implementation of permanent countermeasures against the gravity deformation blocks at the site.

2 Available Data for Recommendation

Prior to this recommendation, we asked NHAI to obtain necessary data by hiring a local consultant. Our requests were explained in the study report submitted to NHAI in our letter[1]. Our requests were incorporated into RFP for hiring a DPR consultant. After the Consultant was hired by NHAI, through discussion among JICA Experts, NHAI, and the Consultant, the requirement was revised and explained in our letter[2].

The requirements are essential for estimating properties of the gravity deformation blocks and considering countermeasure options. Thus these are ordinary and common survey items in Japan.

In India, however, considering capabilities of local consultants and difficulties of the site, some of the TOR might be challenging. For some survey items, the results failed to meet the requirements and inevitably affect quality of this recommendation.

The reasons of failing to meet the requirements are partially due to the delay of the study

---

[1] JXPTHE-001/2019 dated on the 3rd of April, 2019

[2] JXPTHE-068/2019 dated on the 19th of March, 2020

caused by COVID-19 and the difficulties from severe weather and topographic conditions of the site. Considering the fact that such survey items are ordinary in Japan, however, the difference in survey technologies between Japan and India should lie largely behind the failing requirements. By improving the technologies of local consultants for such items that failed to meet the requirements, we think the technologies for survey and investigation of landslide and gravity deformation should be drastically improved in India.

In addition, some items are omitted due to the delay of the study caused by COVID-19, which are explained below.

2.1 Core Boring

Core borings were carried out at the 8 sites out of the planned 10 sites. 2 sites were cancelled due to the delay caused by COVID-19, after considering priority. Some boring locations were shifted due to severe terrain conditions as well as difficulties in land lease issues. The locations of core borings are shown in Figure 2-2 and Appendix-1: Plan View.

The required core recovery ratio is 90 % or more. According to the reports by the local consultant, however, the core recovery ratio was substantially low and only hard rock parts seems to have been recovered.

Improving core recovery ratio will directly improve accuracy of survey. Thus efforts to improve core recovery ratio are worth to be paid. In general, core recovery ratio depends on the following factors:

- Quantity of drilling water to be provided to a core bit
- Pressure of drilling water to be provided to a core bit
- Viscosity and density of drilling water
- Rotation speed of drilling
- Pressure to be provided to a core bit, including weights of drilling rods and tools

To improve core recovery ratio, sensitive control of the aforementioned items is essential. In addition, proper maintenance of drilling tools and periodic supply of consumable core bits are necessary. Proper fixing of a drilling rig on site is also important for stable drilling and loading pressure; both are important factors for core recovery ratio.

To stimulate local consultants to improve core recovery ratio, giving incentive for perfectly high core recovery ratio may be an option. In addition, approving additional payment for adding borehole TV may be effective.

2.2 Groundwater Variation Analysis Chart (GVAC)

Organizing groundwater variation analysis chart (GVAC, refer to 1.1.2(2)(v) of JICA Slope Guideline) using daily records of drilling was a part of the requirements. For all 7 boreholes conducted in the study, GVACs were nicely prepared by the local consultant.

Organizing GVACs is one of the easiest and most reliable methods to grasp groundwater conditions at the time of boring. Thus preparing daily drilling records each working day with information of drilling work including inserted depths of casing and groundwater level before and after the drilling work is essential and required for the study of landslide and gravity deformation.

In this pilot project in NH54, the local consultant learned and practiced how to prepare GVAC very well. We expect such valuable experience by the local consultant will be widely disseminated in India.

2.3 Landslide Monitoring

In the pilot project in NH54, three types of landslide monitoring were conducted; slope stake survey, groundwater level monitoring, and borehole extensometer.

2.3.1 Slope Stake Survey

Slope stake survey was proposed by JICA Experts in a response to the suspension of site work due to COVID-19, including core boring and installation of equipment for groundwater level monitoring and borehole extensometer. The suspension period of 2020 covered the rainy season, in which valuable monitoring data was expected. Thus we proposed slope stake survey to monitor surface displacement caused by the gravity deformation blocks. Slope stake survey is a very simple monitoring method using simple wooden slope stakes installed at the location where expansion of open cracks are expected.

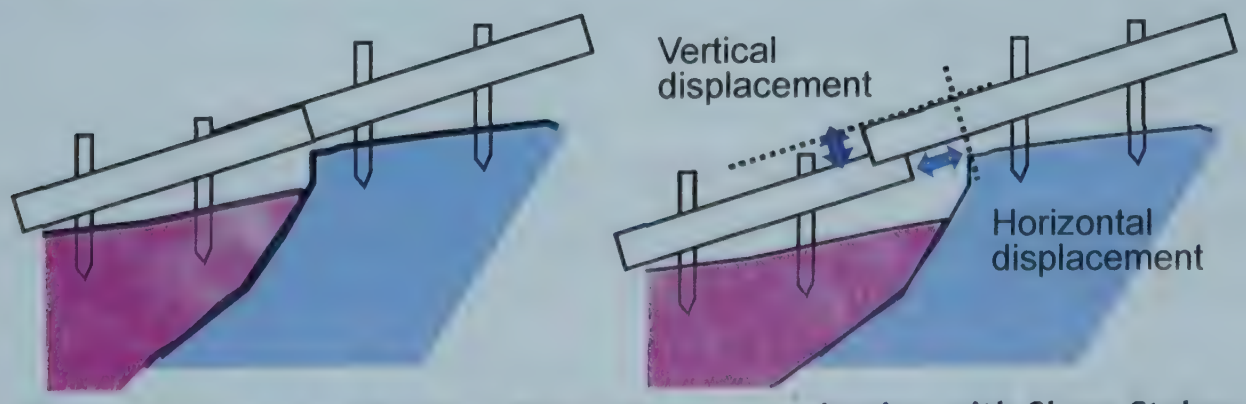

Figure 2-1 Image of Surface Displacement Monitoring with Slope Stakes

JICA Project Team

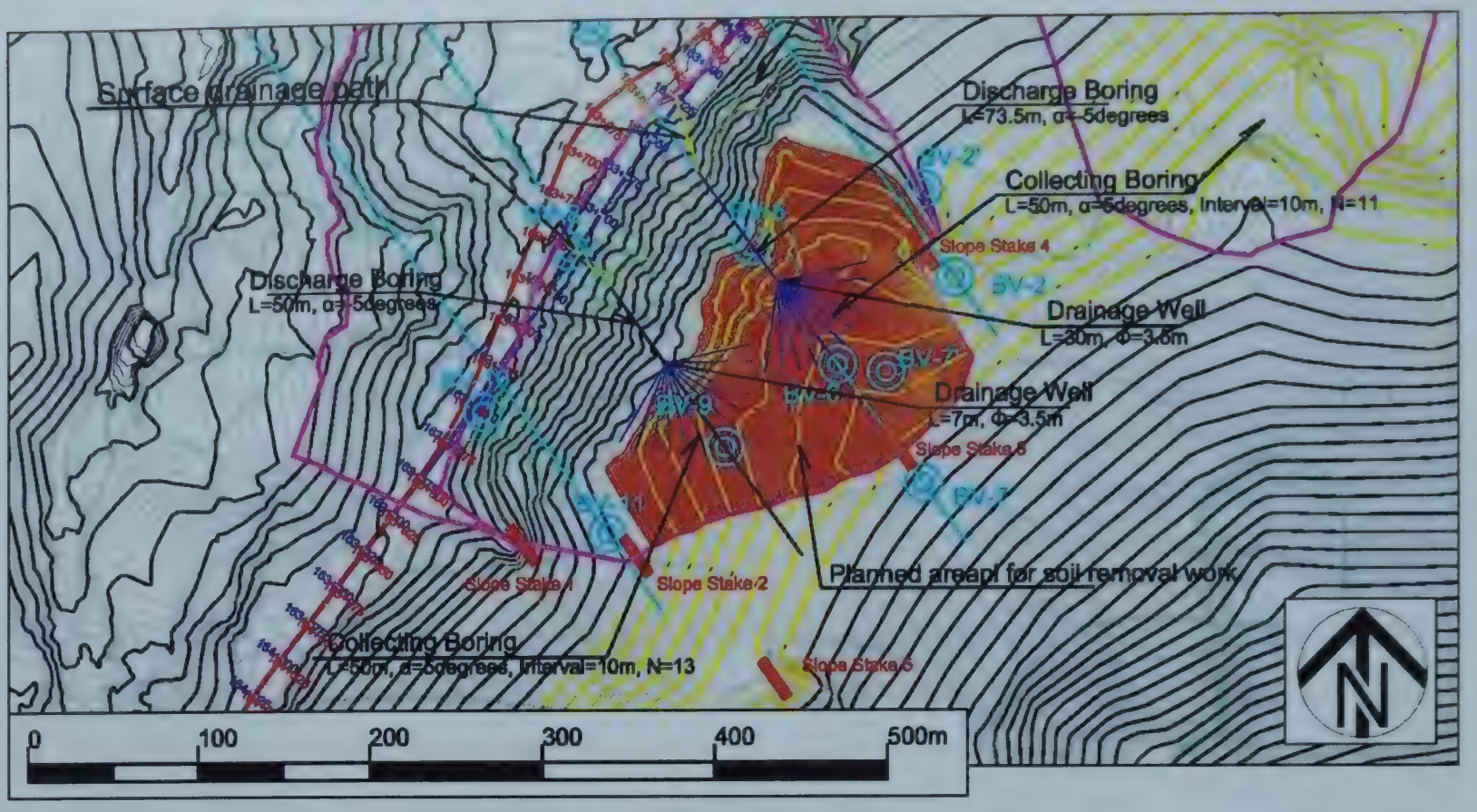

Figure 2-2 Locations of Survey Lines, Boreholes, and Slope Stakes

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Locations of Slope Stakes are shown in Figure 2-2. The slope stakes are planned for the following purposes:

Slope Stake No.1 ~ No.4: To observe deformation / movement of the gravity deformation block surrounded by bold magenta (the target block)

Slope Stake No.5: To observe deformation / movement of the possible gravity deformation block behind the target block.

The monitoring started in June 2020. Figure 2-3 shows the relation among rainfall and displacements of slope stakes since June 2020. The data shown in Figure 2-3 is particularly valuable because it shows a continuous monitoring results throughout rainy and dry seasons.

In Figure 2-3, active displacements of the slope stakes are observed. The displacements show both positive and negative accumulations. Although careful verification is required for the negative accumulations, they may be caused by the structure of slope stakes explained in the later part.

On the other hand, both positive and negative accumulations show simultaneous activation, which often corresponds to a rapid increase of rainfall accumulation.

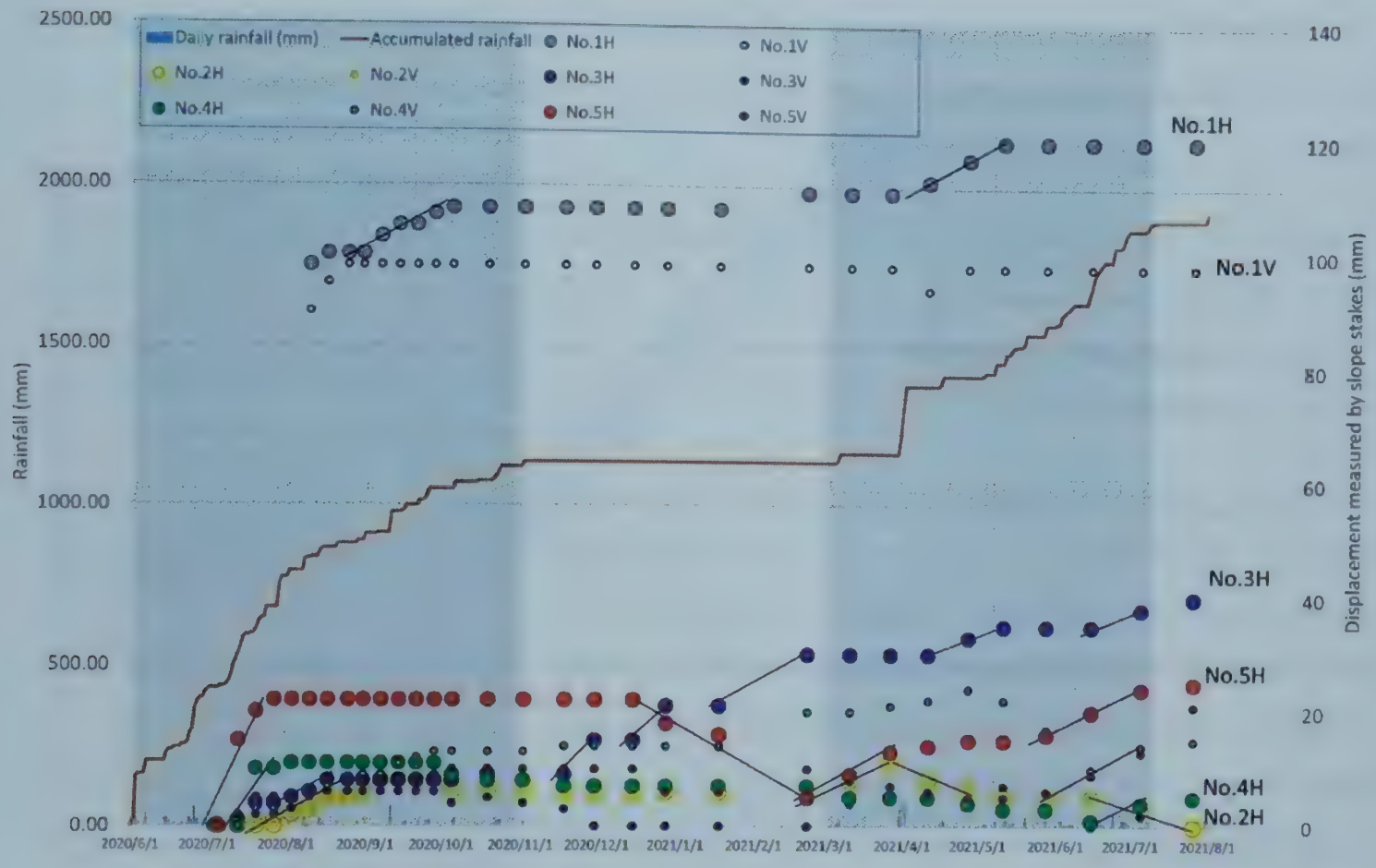

Figure 2-3 Relation among Rainfall and Displacements of Slope Stakes

JICA Project Team

Table 2-1 summarizes characteristics in displacements of the Slope Stakes.

Table 2-1 Characteristics in Displacements of Slope Stakes

Slope Stake No.	Location	Characteristic
No.1	Left side flunk of the target block	Temporary fluctuations in response to rainfall during rainy seasons
No.2	Left side flunk of the target block	Temporary fluctuations in response to rainfall during rainy seasons
No.3	Head of the target block	Intermittent fluctuations throughout the observation period
No.4	Right side flunk of the target block	Temporary fluctuations in response to rainfall during rainy seasons
No.5	Left side flunk of the possible block	Intermittent fluctuations throughout the observation period

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At all slope stakes, displacements in rainy seasons were observed. Thus the target block is evaluated as "active". Among them, the slope stake No.3 at the head of the target block shows intermittent displacements even in the dry season. This special displacements are probably caused by sole movements of the upper part of the target block. But the displacements in dry seasons suggest the block is in extremely dangerous condition.

In addition, the slope stake No.5 was aimed at detecting any displacements caused by the possible block behind the target block. Same as No.3, the slope stake No.5 also shows

intermittent displacements throughout a year even in the dry season. This suggests No.5 may observe the displacements caused by gravity deformation, which proceeds even in dry seasons.

Although the slope stake survey was improvised as an action against the suspension due to COVID-19, it provides substantially important information for the project as mentioned above.

In addition, this slope stake survey also suggests an effective improvement in accuracy of monitoring. Picture 2-1 shows condition of slope stake installation. The flat board installed on the steep slope is supported by 2 posts. With this fixing method, the flat board may rotate around the fixing point with the posts, which enable either post to move separately from others; the distance between the posts may be widened. Figure 2-4 explains deformation of slope stakes when the lower post of the lower stake moves separately in advance. The movement of the lower post makes the upper end of the flat board rotate upward.

Picture 2-1

In the slope stake survey, we observed false negative accumulations[3] possibly because of such deformation of slope stakes shown in Figure 2-4 .

Figure 2-4 Possible Deformation of Slope Stakes Supported by 2 Posts

JICA Project Team

To improve the accuracy of slope stake survey by eliminating false negative accumulation,

[3] Positive accumulation means a downward movement of lower slope stake.

we recommend reinforcement of slope stakes by the manner shown below:

Figure 2-5 Reinforced Slope Stake Installation with Stable Supports

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2.3.2 Groundwater Level Monitoring

Groundwater level monitoring has been done in 4 boreholes of BV-1, BV-3, BV-5 and BV-10. The observation data provided from the local consultant are listed below:

BV-1 : 17th June 2021-1st October 2021, 3times

BV-3 : 15th June 2021-1st October 2021, 6 times

BV-5 : 17th June 2021-1st October 2021, 5 times

BV-10 : 23rd February 2021-1st October 2021, 11 times

Among the provided observation data, the following data was used as input for slope stability analysis:

BV-1 GL-1.87m (17/6/2021)

BV-3 GL-0.00m (13/9/2021)

BV-5 GL-30m (17/6/2021)

2.3.3 Borehole Extensometer

Borehole extensometer monitoring has been done for 2 boreholes of BV-4 and BV-6 at the following depths and observation period:

BV-4 : 12m, 24m, 36m (24th February 2021-1st October 2021, 11 or 12times)

BV-6 : 25m, 35m, 50.5m (20th July 2021-1st October 2021, 4times)

Among the provided observation data, at 36m of BV-4, the latest observation data on the 1st of October, 2021, shows a long extension (-13.15mm =>76.36mm). Further updates are necessary to evaluate the said data to know whether it is significant or not.

On the other hand, at the same depth of 36 m of BV-4, from analysis of daily drilling records and GVAC, stable bedrock is considered to emerge. This coincidence supports credibility of the said observation data as an evidence of slip surface.

2.4 Geophysical Exploration

Resistivity survey in rainy and dry seasons are required when core recovery ratio was less than 90 %. Due to suspension by COVID-19, however, resistivity survey was not conducted in either rainy or dry season.

3 Examination and Analysis

The following steps were made for the recommendation.

1. Estimating depths of slip surface
2. Estimating shape of slip surface
3. Estimating current high water levels (HWL)
4. Estimating design shear strength parameters
5. Estimating initial factor of safety (as a measure against the unstable upper part)
6. Examining groundwater drainage (drainage well)
7. Examining earth removal works

Plan view of the recommended countermeasures are shown in Figure 3-1.

In the following chapters, important points understood through analysis and remarks on design are explained.

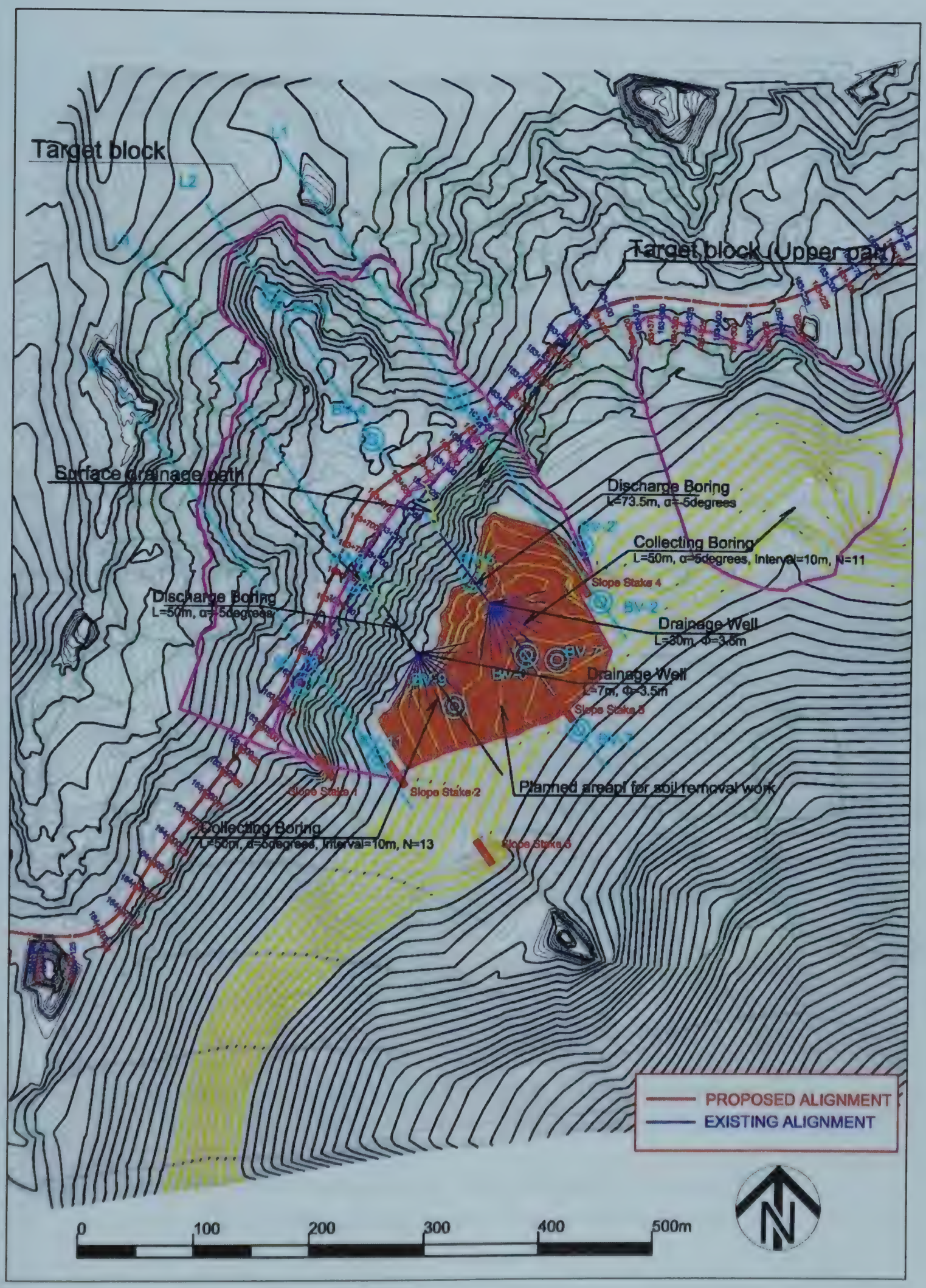

Figure 3-1 Plan View of Recommended Measures

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3.1 Estimating Depths of Slip Surface

This step should be done based on the visual inspection of core samples and observation of underground displacement monitoring. Due to the poor core recovery ratio, however, visual inspection of core samples /photos as well as drilling logs cannot provide entire information for whole depth of borehole.

Based on daily drilling records, groundwater variation analysis chart (GVAC), displacement monitoring with borehole extensometers, and topographic features, therefore, probable depths of slip surface were examined. The results of examination is shown in Appendix-3: Borehole Record Analysis. At the same time, GVAC of each borehole is shown in Appendix-4: Groundwater Variation Analysis.

Drilling speed and leakage ratio are calculated for each drilling run from the daily drilling records in the aim of finding "less loosened zones of bedrock", which is identified by comparing average value of drilling speed and leakage ratio. The depths which show lower values than the average are identified "less loosened" and colored with gradation.

Based on the results, leakage ratio is considered as appropriate factor to identify less loosened zones rather than drilling speed. The lower ends of loosened zones are decided through comprehensive analysis of multiple factors of drilling and groundwater conditions as well as results of landslide monitoring.

Reasons for identification of less loosened zones:

Relatively Less leakage ratio: (refer to Figure 3-2)

Less leakage is considered as less crack and thus less loosened zone as well.

Behavior on GVAC:

In case the following features are detected on GVAC, the corresponding section is considered to have less leakage ratio.

A: Groundwater levels before drilling show similar constant values on consecutive working days.

B: Difference between the groundwater levels after drilling and before drilling of the following working day show little difference.

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER (l/MIN)	RETURNED WATER (L/MIN)	WATER PRESSURE (KpA)	ROTATING SPEED (rpm)	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
07:20	09:40	02:20	140	0	1.5	1.5	1.07142857	nil	Brownish Silty sand with gravel	Greyish	4.64	0	0	1100	4	100
10:10	13:10	03:00	180	1.5	3	1.5	0.83333333	nil	Brownish Silty sand with gravel	Greyish	5.06	0	0	1100	4	100
13:40	15:30	01:50	110	3	4	1	0.90909091	nil	Brownish Silty sand with gravel	Greyish	7.91	0	0	1100	4	100
16:00	17:20	01:20	80	4	5.5	1.5	1.875	nil	Brownish Silty sand with gravel	Greyish	16.75	0	0	1100	4	100
17:40	19:00	01:20	80	5.5	6.5	1	1.25	nil	Brownish Silty sand with gravel	Greyish	16.75	0	0	1100	4	100
06:40	08:50	02:10	130	6.5	7	0.5	0.38461538	0.05	Light Grey. Highly Fractured Rock	Greyish	5	0	0	1100	4	100
09:20	11:30	02:10	130	7	8	1	0.76923077	0.038	Light Grey. Highly Fractured Rock	Greyish	7	0	0	1100	4	100
12:00	14:00	02:00	120	8	9	1	0.83333333	nil	Light Grey. Highly Fractured Rock	Greyish	7.25	0	0	1100	4	100
15:00	17:30	02:30	150	9	10.5	1.5	1	nil	Light Grey. Highly Fractured Rock	Greyish	8.93	0	0	1100	4	100
07:15	10:50	03:35	215	10.5	11.5	1	0.46511628	nil	Light Grey. Highly Fractured Rock	Greyish	3				4	100
11:20	13:00	01:40	100	11.5	12	0.5	0.5	nil	Light Grey. Highly Fractured Rock	Greyish	9				4	100
14:00	16:40	02:40	160	12	13	1	0.625	nil	Light Grey. Highly Fractured Rock	Greyish	5.				4	100
17:00	18:20	01:20	80	13	13.5	0.5	0.625	nil	Light Grey. Highly Fractured Rock	Greyish	16.75	0	0	1100	4	100
07:40	09:30	01:50	110	13.5	14.5	1	0.90909091	0.04	Light Grey. Highly Fractured Rock	Greyish	5.91	3.55	2	1100	4	39.93232
10:10	12:30	02:20	140	14.5	15	0.5	0.35714286	0.03	Light Grey. Highly Fractured Rock	Greyish	6.5	3.9	2	1100	4	40
13:30	16:00	02:30	150	15	16	1	0.66666667	nil	Light Grey. Highly Fractured Rock	Greyish	5.8	3.48	2	1100	4	40
16:30	18:00	01:30	90	16	16.5	0.5	0.55555556	nil	Light Grey. Highly Fractured Rock	Greyish	14.89	8.93	2	1100	4	40.02686
07:20	10:10	02:50	170	16.5	17.5	1	0.58823529	nil	Light Grey. Highly Fractured Rock	Greyish	3.82	2.87	2.3	1100	4	24.86911
10:40	11:20	00:40	40	17.5	18	0.5	1.25	nil	Light Grey. Highly Fractured Rock	Greyish	22.75	17.06	2.3	1100	4	25.01099
07:00	11:30	04:30	270	18	19.5	1.5	[illegible]	0.07	Light Grey. Highly Fractured Rock	Greyish	2.41	2.41	3.5	1100	4	0
12:10	13:00	00:50	50	19.5	20	0.5	[illegible]	[illegible]	Light Grey. Highly Fractured Rock	Greyish	18.2	18.2	3.5	1100	4	0
14:00	16:30	02:30	150	20	21	1	0.66666667		[illegible] Grey. Highly [illegible] Rock	Greyish	5.8	5.8	3.5	900	6	0
17:00	18:40	01:40	100	21	22	1	1					13.4	3.5	900	6	0
06:40	08:40	02:00	120	22	23	1	0.83333333					5.42	3.5	900	6	0
09:10	11:00	01:50	110	23	24	1	0.90909091					8.27	3.45	900	6	0
11:30	13:30	02:00	120	24	25	1	0.83333333					7.25	3.5	1100	4	0
14:20	16:40	02:20	140	25	26.5	1.5	1.07142857					9.57	3.4	1100	4	0
17:10	18:50	01:40	100	26.5	27	0.5	0.5	0.025	Light Grey. Highly Fractured Rock	Greyish	13.4	13.4	3.4	1100	4	0
07:20	09:10	01:50	110	27	28	1	0.90909091	0.08	Light Grey. Highly Fractured Rock	Greyish	5.91	5.91	3.5	1100	4	0
09:50	11:20	01:30	90	28	28.5	0.5	0.55555556	0.06	Light Grey. Highly Fractured Rock	Greyish	10.11	10.11	3.5	1100	4	0
11:50	13:00	01:10	70	28.5	29	0.5	0.71428571	0.12	Light Grey. Highly Fractured Rock	Greyish	12.43	12.43	3.5	1100	4	0
14:00	16:30	02:30	150	29	30	1	0.66666667	0.13	Light Grey. Highly Fractured Rock	Greyish	8.93	8.93	3.56	1100	4	0

Decreasing of leakage ratio

Showing results of Groundwater Variation Analysis Chart (GVAC)

Figure 3-2 Borehole Record Analysis

JICA Project Team

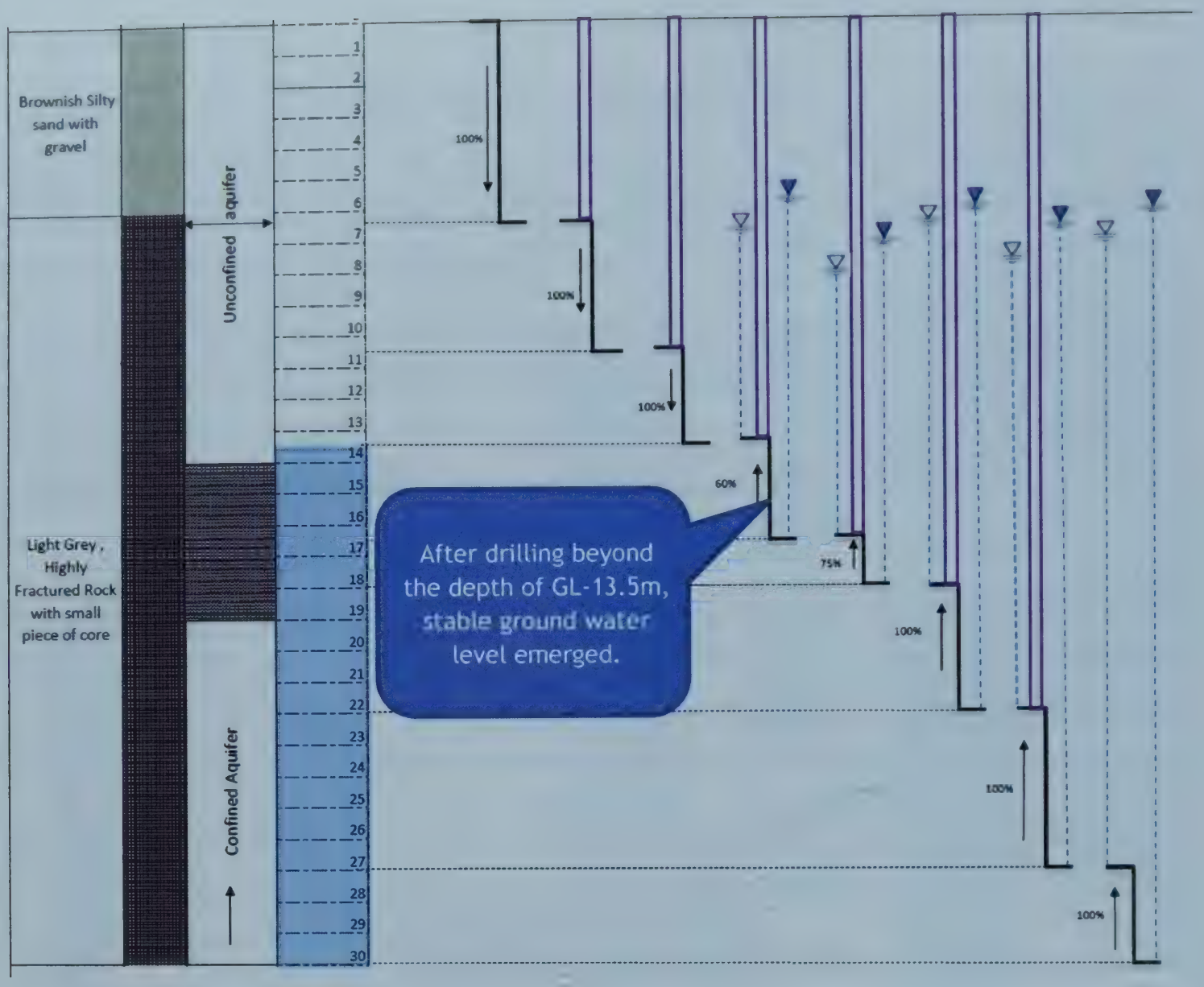

Figure 3-3 Groundwater Variation Analysis Chart (GVAC)

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Stability analysis and examination of countermeasures were carried out on the main survey line of L2, on which boreholes of BV-3, BV-4, BV-5, BV-6, and BV-7' were drilled. The Survey line of L1, L3, and L4 were set as a sub survey line but analyses of L3 and L4 were not conducted due to lack of drilling results. The sub survey line of L1 has 2 boreholes on it but, without L3 and L4, analysis would have been over emphasized on the right side of the target block if 3-D analysis[4] had been done only with L1 and L2.

2-D Analysis of the main survey line was carried out for the recommendation.

The depths of the slip surfaces of each borehole are explained below:

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[4] If sufficient data is available, 3-D slope stability analysis may provide more accurate results when soil removal works are examined.

Table 3-1 Depths of Slip Surfaces

Sub Line L1 : BV-1, BV-2'

Set point	Depth of Slip Surface	Design basis
Toe of landslide	Ground surface	Based on the shape of slip surface projected from Line2
BV-1	GL-15.5m	Borehole record(leakage ratio), GVAC
BV-2'	GL-18m	Borehole record(leakage ratio), GVAC
Head of Landslide	Ground surface	Concave knick point as the head of landslide

Main Line L2 : BV-3, BV-4, BV-5, BV-6, BV-7'

Set point	Depth of Slip Surface	Design basis
Toe of landslide	Ground surface	Concave knick point as the toe of landslide
BV-3	GL-28.74m	Intersection of the line connecting the depth of slip surface at BV-4 and the concave knick point at the toe of landslide
BV-4	GL-36m	Borehole record(leakage ratio), monitoring data (1/10/2021)
BV-5	GL-50m	Depth of the borehole
BV-6	GL-46m	Borehole record(leakage ratio), GVAC
BV-7'	GL-18m*	Borehole record(leakage ratio), GVAC *Not used for analysis because it is projected on L2
Head of Landslide	Ground surface	Concave knick point as the head of landslide

Sub Line L4 : BV-10

Set point	Depth of Slip Surface	Design basis
BV-10	GL-34m	Borehole record(leakage ratio), GVAC

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3.2 Estimating Shape of Slip Surface

The slip surface on the main survey line of L2 was decided by connecting the depths of slip surface decided in the previous section as shown below:

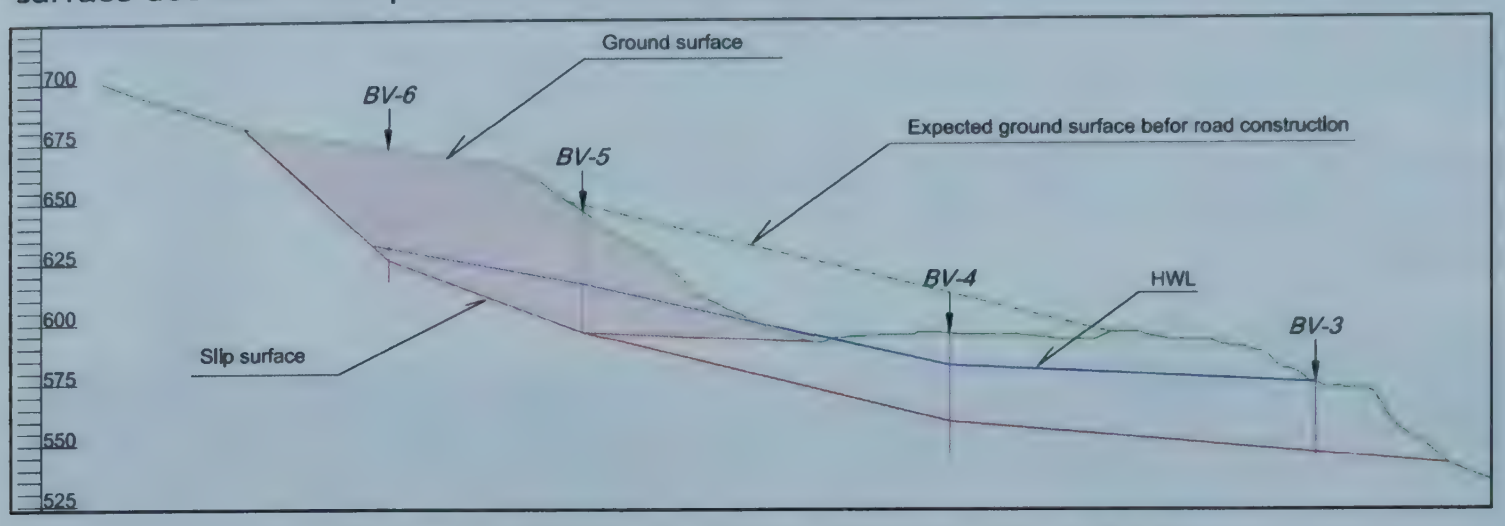

Figure 3-4 Slip Surface on Main Survey Line of L2

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The slip surface shows a non-circular arc shape. In Figure 3-4, possible ground surface before road widening is shown in the green broken line. From the shape of the current ground surface and the slip surface, which is shown in the red line, it is highly likely that the excavation for road widening removed a substantial weight in the middle of the existing landslide block and caused instability of the entire block. Considering the extremely unstable condition of the cut slope, the upper part of the landslide block on hill side of NH54 (shown in pale red in Figure 3-4) can slide alone, separated from the entire block.

On the other hand, in the past field survey, significant movement was not confirmed at the lower slope. The latest monitoring result suggested a sign of a substantial displacement, the entire block may be activated.

3.3 Estimating Current High Water Levels

A High Water Level (HWL) for stability analysis was set refereeing to the groundwater level observed during drilling of each borehole as well as results of groundwater level monitoring. Although HWL was set for 7 boreholes as shown below, slope stability analysis was made on the main survey line of L2 only.

Table 3-2 Setting High Water Level (HWL)

Sub Line L1 : BV-1, BV-2'

Point	HWL	Design basis
BV-1	GL-1.87m	Monitoring data (17/6/2021)
BV-2'	GL-7m	GVAC

Main Line L2 : BV-3, BV-4, BV-5, BV-6, BV-7'

Point	HWL	Design basis
BV-3	GL-0.00m	Monitoring data (13/9/2021)
BV-4	GL-13m	GVAC
BV-5	GL-30m	Monitoring data (17/6/2021)
BV-6	GL-41m	GVAC
BV-7'	GL-15m*	GVAC *Not used for analysis because it is projected on L2

Sub Line L4 : BV-10

Point	HWL	Design basis
BV-10	GL-0.47m	Monitoring data (5/7/2021)

3.4 Estimating Design Shear Strength Parameters

Based on the following features, the target block is considered as a repetitive landslide with clay (Jisuberi in Japanese) rather than a block of gravity deformation of bedrock.

- As understood from Figure 3-4, excluding the section below the head scarp, the slip surface of the target block shows relatively gentle inclination of 4 ~20 degrees.
- The geological strata is highly fractured shale, and
- Interbedded clay was observed during bore boring.

Referring to landslide studies in Japan, internal friction angle of slip surface of repetitive landslides with clay falls in a range between 5 and 16 degrees[5].

The slip surface shows a non-circular arc shape while running through bedrock. Thus Janbu

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[5] Mayumi T, Shibazaki T, Yamazaki T(2003) : Evaluation on Shear Strength of Slip Surface by Shear Strength Test on Slip Surfaces, JISUBERI Vol.40, No.4, pp.15～24

Method was employed as a formula for slope stability analysis.

Unit weight of landslide mass was set as 23 kN/m3, referring to the minimum range of the empirical values for shale of 23 ~ 27 kN/m3 reported in a landslide study in Japan[6]. The minimum value was adopted due to highly fractured condition of landslide mass, which suggests increased void space compared to an intact status.

Design shear strength was set through back analysis assuming that the following conditions form a limited equilibrium status of Fs=1.0:

- Possible ground surface before road widening was set as the surface of the slope.
- HWL observed in the groundwater level monitoring was set as groundwater level.

For repetitive landslide with clay, shear strength on slip surface is considered as residual strength, while the cohesive factor decreases to almost zero, based on the theory of soil mechanics. Thus a probable internal friction angle was calculated through back analysis with the condition of cohesion as zero.

- ✓ Applied formula: Janbu Method
- ✓ Terrain shape condition: ground surface before road widening, assuming a potential landslide lied before the widening
- ✓ Groundwater condition: the highest water level during drilling and monitoring.
- ✓ Unit weight: empirical value of shale referring to a study in Japan

The design shear strength of slip surface calculated through back analysis is as follows:

- ✓ c'=0, ϕ' =16.3977 degrees

3.5 Estimating Initial Factor of Safety (as a Measure against the Unstable Upper Part)

As explained in 3.2 Estimating Shape of Slip Surface, the landslide was activated when a substantial excavation for the road widening of NH54 was implemented. After the activation, the upper part of the landslide significantly lost its stability and thus started to slide alone apart from the entire block. The unstable upper part pushes the roadside cut slope of NH54 from behind and make it unstable as well.

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[6] Kojima K, Nakao K (1995):Basic and Practice of Engineering Geology, Kajima Publishing

The unstable upper part may stimulate activities of the possible gravity deformation blocks behind. In this pilot project, unfortunately the study for the possible gravity deformation blocks was not sufficiently conducted. As explained in 2.3.1 Slope Stake Survey, however, the intermittent displacements throughout rainy and dry seasons were observed at the possible gravity deformation block behind the unstable upper part of the landslide. If the unstable upper part significantly slide, therefore, the possible gravity deformation block can loose it stability and may cause further landslides or failures, or even a deep-seated landslide.

As explained above, besides assuring the stability of the entire landslide, improving the stability of the unstable upper part is particularly important.

Estimating initial factor of safety for design consideration was done in the following process:

1) Setting Factor of Safety Fs=1.0 with the ground surface before road widening and HWL estimated in 3.3 Estimating Current High Water Levels[7].
2) Estimating Initial Factor of Safety for Design for the entire landslide block with the current ground surface.
3) Estimating Initial Factor of Safety for Design for the upper part of the landslide.

Table 3-3 Initial Factor of Safety for Design

Target blocks	Initial Factor of Safety
Critical Status before Road Widening	1.000
Entire Landslide Block	0.9195
Upper Part of the landslide	0.7627

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As understood from Table 3-3, due to the excavation, the entire block as well as the upper part significantly lost its stability and the upper part started to move separately from the entire block.

Thus the countermeasures should be so designed that both entire and upper part of the landslide are stabilized. In the design procedure explained in the later chapters, the initial factor of safety shown in Table 3-3 is improved by adding countermeasures to meet a Design Factor of Safety (DFs), for both entire and the upper part of the landslide.

---

[7] In ideal condition, HWL should be the critical water level (CWL) before the road widening, which is not available after the widening. Thus the highest water levels observed during the survey were used as HWL.

3.6 Examining Groundwater Drainage (Drainage Well)

In the design of countermeasures against landslides, the effects by horizontal drainage boring are considered as a certain meters decrease of groundwater level in slope stability analysis, which positively affects a factor of safety.

In Japan, the effects by horizontal drainage borings from the surface are considered as a 3 meters decrease of groundwater level, whereas those by a drainage well, which installs horizontal drainage borings from subsurface, are considered as a 5 meter decrease. The reason why the effects by a drainage well are considered larger is that the horizontal drainage borings from a drainage well can be placed deeper along the targeted slip surface of a repetitive landslide with clay. Thus effective drainage sections of drainage borings from a drainage well are longer than those of horizontal drainage boring from the surface as show in the figure below.

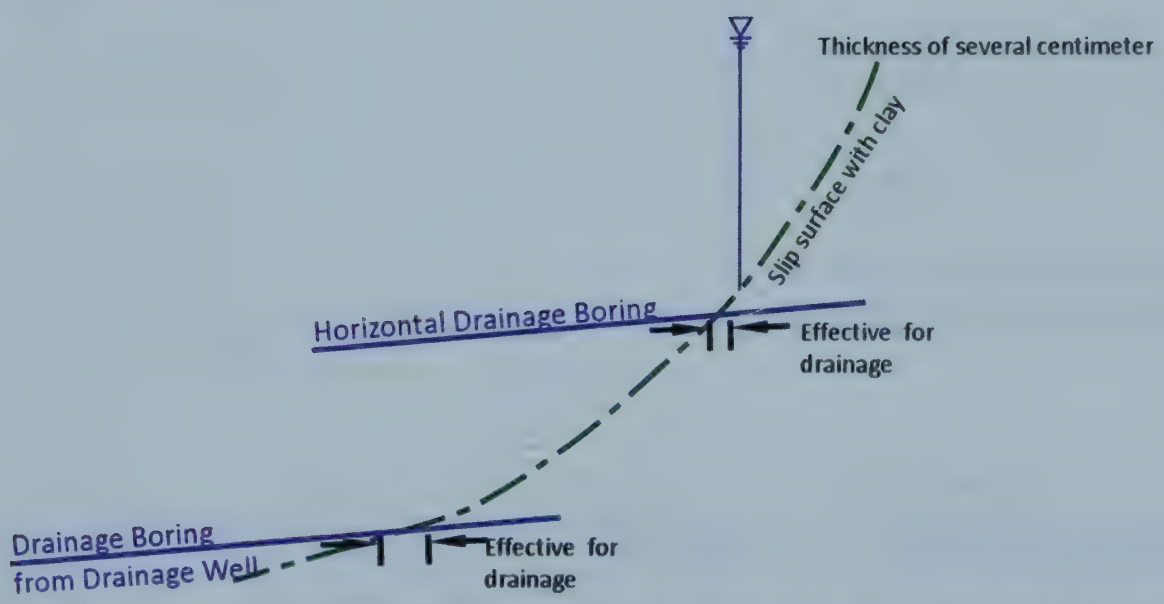

Figure 3-5 Effects of Drainage Boring from Surface and Drainage Well at Pilot SIte

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In the pilot project site where the slip surface running deep subsurface, horizontal drainage borings from the surface are not sufficient to effectively release pore water pressure on the slip surface. In this recommendation, therefore, horizontal drainage borings from drainage wells are considered.

In design, the effects by drainage wells are considered as a 5 meters decrease of groundwater level, which is same as an empirical value in Japan. The effects by drainage wells, therefore, should be verified through groundwater level monitoring.

The monitoring should start at least 1 year in advance before the construction of the drainage wells so that the effects by drainage wells are properly verified. In case the observed decrease of groundwater level fails to meet the expected decrease in the design, additional horizontal drainage borings and drainage wells should be planned and

implemented.

Table 3-4 Increase of Factor of Safety by Drainage Well

Target blocks	Initial Factor of Safety	Improved Factor of Safety
Entire Landslide Block	0.9195	0.9814
Upper Part of the landslide	0.7627	0.8062

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In Japan, for design of countermeasures against repetitive landslide with clay, the maximum increase of factor of safety by groundwater control works including drainage well is limited as 5 % or less. Considering additional implementation of horizontal drainage borings and drainage wells based on the verification by groundwater level monitoring, in this recommendation, a limitation for the maximum increase of factor of safety by drainage well was not set.

In Appendix-6: Plan and Profile of Recommendation, image of drainage wells arrangements are illustrated along with that of earth removal works explained in the next section.

In Appendix-7: Image of Drainage Well, example drawings for a drainage well are attached.

Chapter 2.7.2 (4) and Chapter 3.1.4 of JICA Slope Guideline provide further information regarding design and construction of drainage well. The calculation for the drainage well on L2 is attached as Appendix-8.

3.7 Examining Earth Removal Works

To stabilize a landslide block, direct weight removal should work effectively. The effective locations for weight reduction is, however, limited to a narrow section where a slip surface has a steep inclination as an extension from a head scarp.

On the other hand, excessive earth removal works may destabilize the possible gravity deformation block behind, by removing counterweight against the deformation. Thus earth removal works should be designed as small as possible while securing necessary improvement of factor of safety.

Considering the aforementioned limitation, effectiveness and feasibility of earth removal works were examined by considering step by step cutting works with a height of 5 meters for one step cutting.

Table 3-5 explains the examination results of earth removal works. With the Cut_5, the improved factor of safety for the entire block reaches Fs=1.2. Thus the earth removal up to the Cut_5 is considered as the maximum volume of cutting to meet the purpose of

securing stability of the landslide.

With the Cut_5, the improved factor of safety for the upper part barely reaches Fs=1.1. Considering the said limitation, however, further cutting should be avoided in fear of destabilizing the possible gravity deformation block behind.

Table 3-5 Increase of Factor of Safety by Earth Removal Works

Target blocks	Step of Cutting	Improved Factor of Safety by Drainage Well	Improved Factor of Safety by Earth Removal
Entire Landslide Block	Cut_1	0.9814	1.0475
	Cut_2		1.0813
	Cut_3		1.1172
	Cut_4		1.1558
	Cut_5		1.1966
Upper Part of the landslide	Cut_1	0.8062	0.8860
	Cut_2		0.9279
	Cut_3		0.9760
	Cut_4		1.0328
	Cut_5		1.1019

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Surface and subsurface displacement monitoring should be required for verifying the effects of the earth removal. Monitoring should start before the construction. In case the upper part is not stabilized even after earth removal works and drainage wells are completed, additional countermeasures by prevention works including ground anchor works should be considered.

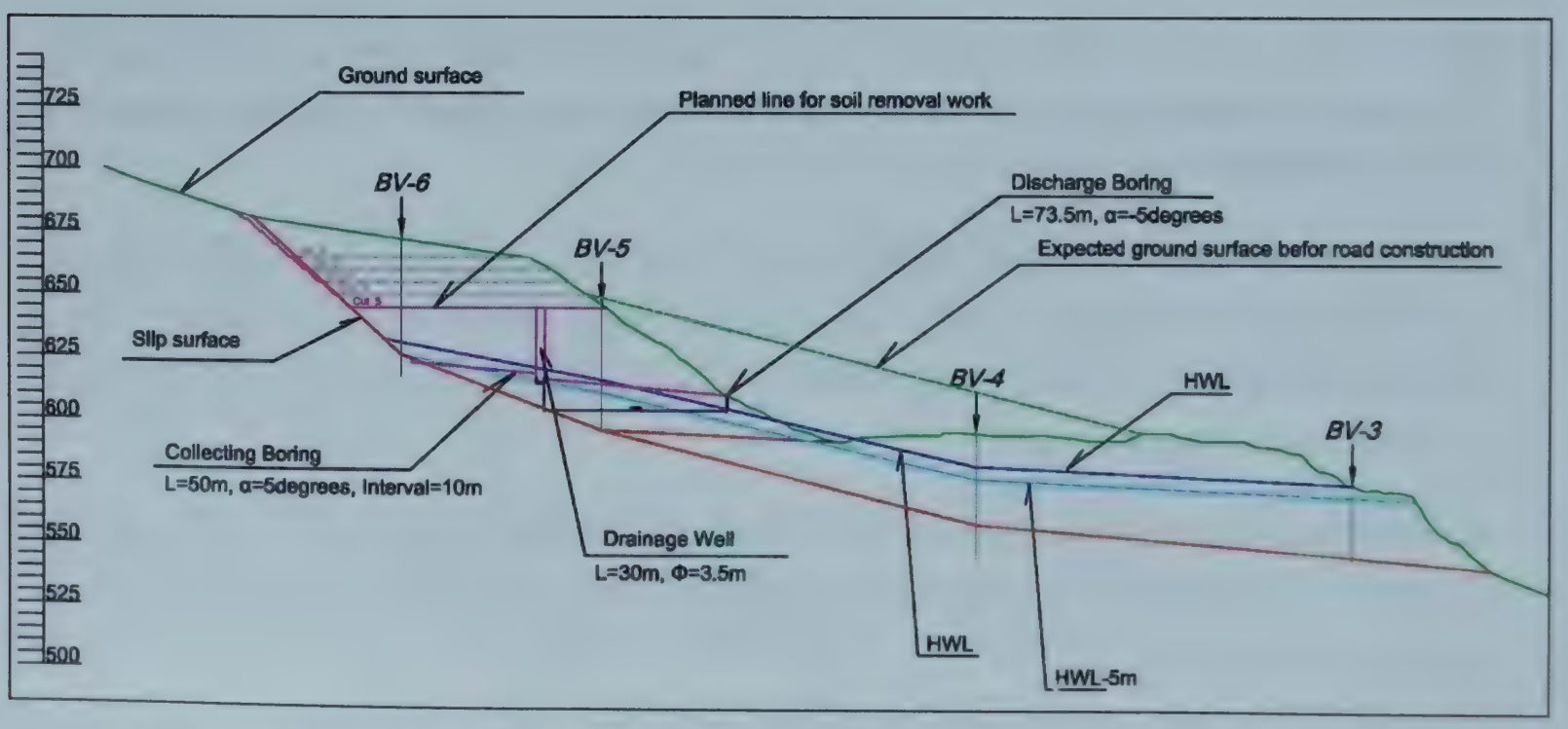

Figure 3-6 Arrangements of Recommended Measures on L2 Profile

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For the surface of cut slope to be emerged by the earth removal works, proper slope

protection and erosion control should be implemented, including applying vegetation on slope surface and arranging surface drainage system.

4 Conclusion

Remarks on implementation and future improvement are summarized below:

4.1 Improvement of Core Boring

In this recommendation, estimation of depths of slip surfaces was done with daily drilling records and groundwater variation analysis chart (GVAC). Thanks to the detailed records of drilling condition recorded by the drilling operators, the estimated depths of slip surfaces were considered appropriate though the comprehensive analysis on the main and sub survey lines of L2 and L1.

The total lengths of the recovered samples through core boring were shorter than the drilling depths, however. With the low core recovery ratio, core samples obtained through core borings do not represent the whole depths of geological conditions. Comparing core samples with a high recovery ratio of more than 95%, the core samples obtained in the pilot project lack substantial information. Thus accuracy of the estimated depths of slip surfaces should have certain amounts of errors.

To improve accuracy in estimating depths of slip surfaces, all core sampling for whole depths of drilling with a core recovery ratio of 100% is necessary. Further improvement of core boring technique is expected, therefore.

In addition, improvement of subsurface displacement monitoring utilizing boreholes, including pipe strain gauge and borehole inclinometer, are essential as well.

4.2 Conducting Canceled Survey

In the plan view of the recommendation attached to Appendix6, arrangements of a drainage well on sub survey line of L3 is illustrated, which should be understood temporary plan.

If NHAI wants to implement the drainage well on L3, similar surveys conducted on the main survey line of L2 should be done for L3 as well, including core boring and landslide monitoring. The details of the drainage well on L3 should be examined on the profile on L3 prepared through the survey.

With detailed information on L3, 3-D analysis may be done along with the existing information on L2, which may improve accuracy of 2-D analysis on L2 and reduce necessary

quantities of the countermeasure works.

For the time being, the drainage well on L2 should be implemented first and effects of the works can be verified through landslide monitoring. In case the landslide is not sufficiently stable with the implemented works, additional works should be implemented as step construction.

4.3 Possible Gravity Deformation Block Behind

In this recommendation, massive earth removal works are planned at the head of the landslide. Earth removal works are effective measures against gravity deformation and repetitive landslide with clay, with direct reduction of weight from the head to improve stability of landslide. Thus, along with drainage wells, earth removal works should greatly contribute to the reduction of disaster risks on NH54.

On the other hand, if another risk of landslide / mass movement lies behind the target block, the massive earth removal works reduce stability of the possible block behind.

Among the slope stakes monitoring in the pilot project, the slope stake No.5 monitors displacements by the possible gravity deformation block behind. Thus we strongly recommend the observation of the slope stake No.5 should be continued after the completion of the countermeasure works. By keeping observation of the slope stake No.5 with the recommended reinforcement in 2.3.1, basis for considering effective options against the possible gravity deformation block can be obtained.

For the possible gravity deformation block behind the landslide, deep core boring should be conducted along with installing subsurface displacement monitoring equipment into the borehole and keeping observation of it. Such deep core boring can be done at the original location of BV-7, which was shifted to BV-7' due to the severe site condition. With the proposed deep core boring and displacement monitoring, risk of the possible gravity deformation will be adequately evaluated. The displacement monitoring may be used as a sensor for early warning system for NH54.

For the surface of cut slope to be emerged by the earth removal works, as described in 3.7, at the time of construction, proper slope protection and erosion control should be implemented, including applying vegetation on slope surface and arranging surface drainage system.

In longer term, after stabilizing the possible gravity deformation behind or confirming it is stable even after the construction of the earth removal, the surface should be protected

by concrete crib works with vegetation within the frames.

4.4 Slope Failure on Roadside Cut Slope of NH54

Considering the accumulated displacement observed in the slope stake survey and monitoring at BV-4, the entire landslide block is considered destabilized. In addition, the upper part of landslide is more unstable.

Under this condition, without implementing the countermeasure works recommended in this paper, slope protection applied to the cut slope of NH54 will simply be damaged again.

Until both the entire block and the upper part of the landslide are stabilized, slope protection with rigid structure is not recommended. For the time being, erosion and weathering control works on the slope and temporary protection wall along NH54, which can prevent collapse and rock fall from the cut slope, are recommended.

After the landslide is completely stabilized, slope protection with rigid structure should be designed, including concrete crib work, rock bolts / soil nailing, and shotcrete.

4.5 Fence on Drainage Well

Since a drainage well is composed of a very deep shaft, which is potentially dangerous for the people living around the facility, protection to prevent intrusion by a third party should be well prepared.

The top of a drainage well must be closed with a lid with locks. A drainage well should be protected by a fence surrounding well platform to deter intrusion by a third party.

Figure 4-1 Fencing and Protection for Drainage Well

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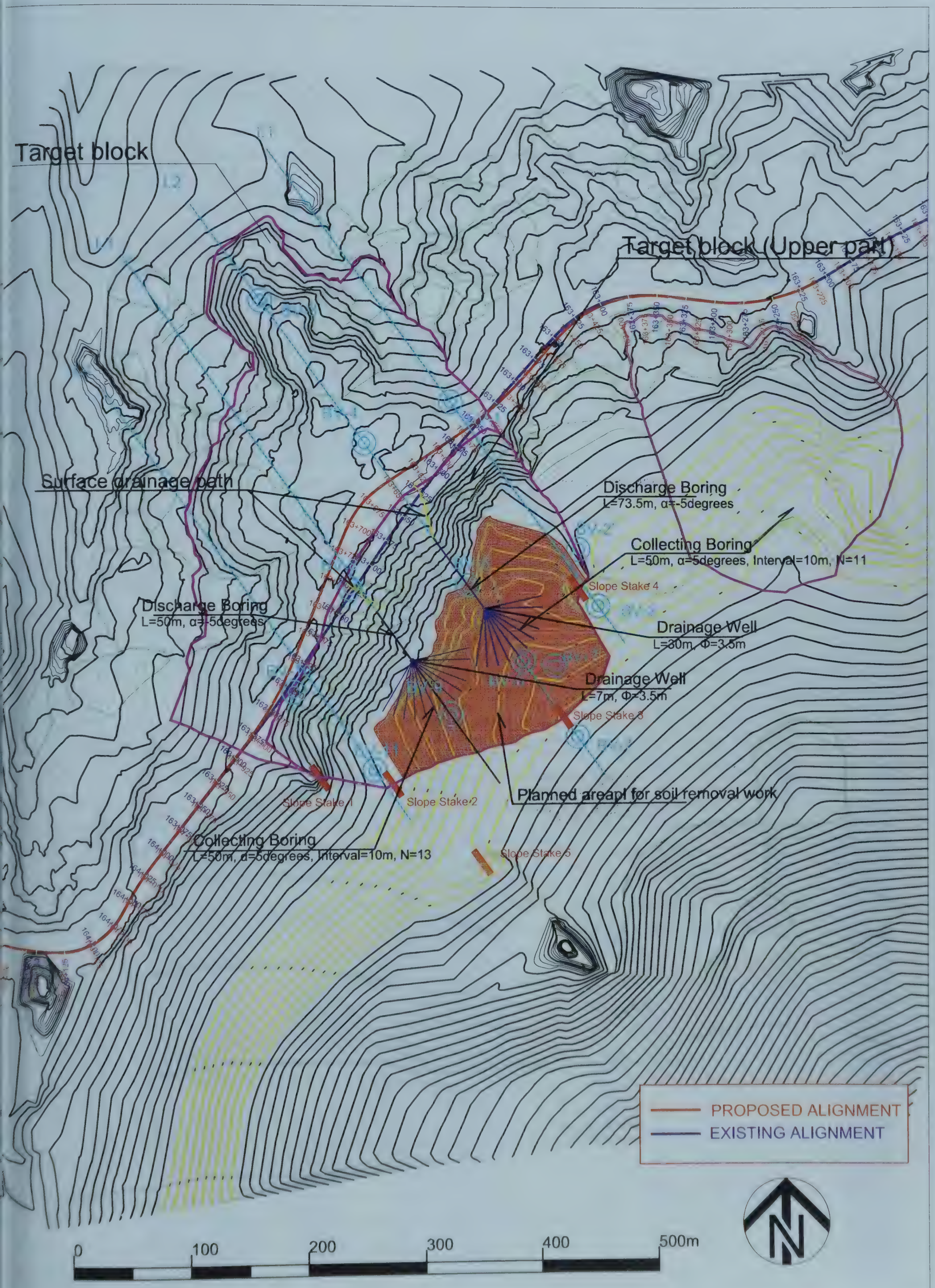
Appendix 1

Locations of Boreholes and Survey Lines


Target block
Target block (Upper part)
Surface drainage path
Discharge Boring
L=73.5m, α=-5degrees
Collecting Boring
L=50m, α=5degrees, Interval=10m, N=11
Slope Stake 4
Discharge Boring
L=50m, α=-5degrees
Drainage Well
L=30m, Φ=3.5m
Drainage Well
L=7m, Φ=3.5m
Slope Stake 3
Planned areapl for soil removal work
Slope Stake 1
Slope Stake 2
Collecting Boring
L=50m, α=5degrees, Interval=10m, N=13
Slope Stake 5
PROPOSED ALIGNMENT
EXISTING ALIGNMENT
0
100
200
300
400
500m
N

Appendix 2

Borehole Record Analysis

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 1 Line-1

Date: 22/3/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
12:20	14:00	1:40	100	0	1.5	1.5	1.5	nil	Brownish Silty sand with gravel	Greyish	6.5	0	0	1100	4	100
14:30	16:00	1:30	90	1.5	2.5	1	1.11111111	nil	Brownish Silty sand with gravel	Greyish	9.1	0	0	1100	4	100
16:20	17:30	1:10	70	2.5	3	0.5	0.71428571	nil	Brownish Silty sand with gravel	Greyish	9.67	0	0	1100	4	100
17:50	19:00	1:10	70	3	4	1	1.42857143	nil	Brownish Silty sand with gravel	Greyish	14.89	0	0	1100	4	100
7:00	9:20	2:20	140	4	5	1	0.71428571	nil	Light Grey, Highly Fractured Rock	Greyish	11.57	0	0	1100	4	100
10:10	13:30	3:20	200	5	5.5	0.5	0.25	nil	Light Grey, Highly Fractured Rock	Greyish	6.75	0	0	1100	4	100
14:00	15:10	1:10	70	5.5	6.5	1	1.42857143	nil	Light Grey, Highly Fractured Rock	Greyish	19.38	0	0	1100	4	100
15:40	18:10	2:30	150	6.5	7	0.5	0.33333333	0.035	Light Grey, Highly Fractured Rock	Greyish	12.18	0	0	1100	4	100
10:00	13:00	3:00	180	7	8	1	0.55555556	nil	Light Grey, Highly Fractured Rock	Greyish	7.78	0	0	1100	4	100
14:10	15:40	1:30	90	8	8.5	0.5	0.55555556	nil	Light Grey, Highly Fractured Rock	Greyish	9.07	0	0	1100	4	100
16:10	18:10	2:00	120	8.5	9	0.5	0.41666667	nil	Light Grey, Highly Fractured Rock	Greyish	15.09	0	0	1100	4	100
8:00	10:30	2:30	150	9	10	1	0.66666667	0.07	Light Grey, Highly Fractured Rock	Greyish	10	0	0	1100	4	100
11:40	13:20	1:40	100	10	11.5	1.5	1.5	nil	Light Grey, Highly Fractured Rock	Greyish	12.1	0	0	1100	4	100
14:00	16:40	2:40	160	11.5	12.5	1	0.625	nil	Light Grey, Highly Fractured Rock	Greyish	11.81	0	0	1100	4	100
17:20	19:30	2:10	130	12.5	13	0.5	0.38461538	nil	Light Grey, Highly Fractured Rock	Greyish	8.08	0	0	1100	4	100
7:30	9:30	2:00	120	13	13.5	0.5	0.41666667	nil	Light Grey, Highly Fractured Rock	Greyish	9.58	0	0	1100	4	100
10:10	12:20	2:10	130	13.5	14.5	1	0.76923077	0.06	Light Grey, Highly Fractured Rock	Greyish	11.15	0	0	1100	4	100
13:30	15:40	2:10	130	14.5	15	0.5	0.38461538	0.025	Light Grey, Highly Fractured Rock	Greyish	9.31	0	0	1100	4	100
16:10	18:40	2:30	150	15	15.5	0.5	0.33333333	0.04	Light Grey, Highly Fractured Rock	Greyish	7.47	0	0	1100	4	100
7:00	9:30	2:30	150	15.5	16	0.5	0.33333333	0.038	Light Grey, Highly Fractured Rock	Greyish	5.4	3.78	2	1100	4	30
9:50	11:50	2:00	120	16	17	1	0.83333333	0.057	Light Grey, Highly Fractured Rock	Greyish	9.83	6.88	2	1100	4	30.01017
12:10	14:00	1:50	110	17	17.5	0.5	0.45454545	0.038	Light Grey, Highly Fractured Rock	Greyish	8.09	5.66	2.35	1100	4	30.03708
15:40	17:40	2:00	120	17.5	18.5	1	0.83333333	nil	Light Grey, Highly Fractured Rock	Greyish	9.5	6.65	2.5	1100	4	30
7:40	11:00	3:20	200	18.5	19.5	1	0.5	0.05	Light Grey, Highly Fractured Rock	Greyish	7.1	6.04	2.5	1100	4	14.92958
11:35	13:30	1:55	115	19.5	20.5	1	0.86956522	0.06	Light Grey, Highly Fractured Rock	Greyish	10.87	9.24	2.5	1100	4	14.9954
14:10	16:30	2:20	140	20.5	22	1.5	1.07142857	0.08	Light Grey, Highly Fractured Rock	Greyish	12	10.2	3.2	1100	4	15
16:50	18:30	1:40	100	22	22.5	0.5	0.5	0.035	Light Grey, Highly Fractured Rock	Greyish	10.2	8.67	3.4	1100	4	15
7:00	10:10	3:10	190	22.5	23.5	1	0.52631579	0.03	Light Grey, Highly Fractured Rock	Greyish	5.26	5.26	3.5	1100	4	0
11:00	14:00	3:00	180	23.5	25	1.5	0.83333333	0.12	Light Grey, Highly Fractured Rock	Greyish	5	5	3.5	1100	4	0
14:40	15:50	1:10	70	25	25.5	0.5	0.71428571	0.065	Light Grey, Highly Fractured Rock	Greyish	10	10	3.6	1000	6	0
16:20	17:20	1:00	60	25.5	26.5	1	1.66666667	0.13	Light Grey, Highly Fractured Rock	Greyish	10.83	10.83	3.6	1000	6	0
17:50	19:10	1:20	80	26.5	27	0.5	0.625	0.081	Light Grey, Highly Fractured Rock	Greyish	13.13	13.13	3.6	1000	6	0
7:30	11:10	3:40	220	27	28	1	0.45454545	0.05	Light Grey, Highly Fractured Rock	Greyish	6.82	6.82	3.4	1100	4	0
11:40	13:00	1:20	80	28	28.5	0.5	0.625	0.045	Light Grey, Highly Fractured Rock	Greyish	8.13	8.13	3.5	1100	4	0
0:00	13:00	3:00	180	28.5	29.5	1	0.55555556	0.04	Light Grey, Highly Fractured Rock	Greyish	7.22	7.22	3.7	1100	4	0
3:40	15:20	1:40	100	29.5	30	0.5	0.5	0.05	Light Grey, Highly Fractured Rock	Greyish	8	8	3.7	1100	4	0
6:10	18:50	2:40	160	30	31	1	0.625	0.175	Light Grey, Highly Fractured Rock	Greyish	7.19	7.19	3.7	1100	4	0

7:30	11:10	3:40	220	31	32	1	0.45454545	0.09	Dark Grey, Highly Fractured Rock	Greyish	6.59	6.59	3.75	1000	6	0
11:40	13:30	1:50	110	32	32.5	0.5	0.45454545	0.12	Dark Grey, Highly Fractured Rock	Greyish	6.55	6.55	3.75	1000	6	0
14:00	15:10	1:10	70	32.5	33	0.5	0.71428571	0.05	Dark Grey, Highly Fractured Rock	Greyish	9.29	9.29	3.75	1000	6	0
7:15	11:10	3:55	235	33	34	1	0.42553191	0.09	Dark Grey, Highly Fractured Rock	Greyish	6.04	6.04	3.7	1100	4	0
11:40	13:30	1:50	110	34	34.5	0.5	0.45454545	0.045	Dark Grey, Highly Fractured Rock	Greyish	7.36	7.36	3.75	1100	4	0
14:10	16:20	2:10	130	34.5	35.5	1	0.76923077	0.125	Dark Grey, Highly Fractured Rock	Greyish	7.23	7.23	3.8	1100	4	0
17:00	18:50	1:50	110	35.5	36.5	1	0.90909091	0.145	Dark Grey, Highly Fractured Rock	Greyish	7.91	7.91	3.8	1100	4	0
7:30	9:40	2:10	130	36.5	37.5	1	0.76923077	0.165	Dark Grey, Highly Fractured Rock	Greyish	7.31	7.31	3.85	1100	4	0
10:20	13:00	2:40	160	37.5	38.5	1	0.625	0.095	Dark Grey, Highly Fractured Rock	Greyish	7.44	7.44	3.85	1100	4	0
14:00	15:20	1:20	80	38.5	39	0.5	0.625	0.045	Dark Grey, Highly Fractured Rock	Greyish	8.5	8.5	3.85	1100	4	0
15:50	18:10	2:20	140	39	40	1	0.71428571	0.075	Dark Grey, Highly Fractured Rock	Greyish	7.71	7.71	3.85	1100	4	0
							0.6984291	0.0733								43.3328

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-15.5 m and deeper, the goundwater levels before drilling were observed as stable. In addition, leackage of drilling water was improved after the drilling of GL-15.5m and deeper as well. Thus at the depth of GL-15.5 m and deeper, it is highly likely that less loosened bedrock lies. The depths of slip surface was, threfore, set at GL-15.5 m.

The highest groundwater level during the groundwater monitoring was observed at GL-1.87 m on the 17th of June, 2021. Even though it is the initial value of the monitoring, GL-1.87 m was set as HWL.

Daily Drilling Operation Report

Project Name: UNSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 2 Line-1

Date: ..... from 29/8/2021 to 10/9/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
10:00	12:30	2:30	150	0	1	1	0.66666667	nil	Silty gravel	Greyish	5	0	0	1100	4	100
13:30	15:40	2:10	130	1	2.5	1.5	1.15384615	nil	Silty gravel	Greyish	5.08	0	0	1100	4	100
16:30	18:10	1:40	100	2.5	3	0.5	0.5	nil	Silty gravel	Greyish	4.6	0	0	1100	4	100
8:00	10:20	2:20	140	3	4.5	1.5	1.07142857	0.35	Grey highly fractured rock with boulder	Greyish	5.5	0	0	1100	4	100
12:10	14:10	2:00	120	4.5	6	1.5	1.25	0.2	Boulder	Greyish	5.2	0	0	1100	4	100
15:20	16:50	1:30	90	6	7.5	1.5	1.66666667	0.075	Boulder	Greyish	4.9	0	0	1100	4	100
17:20	18:10	0:50	50	7.5	8	0.5	1	0.05	Grey highly fractured rock	Greyish	4.85	0	0	1100	4	100
8:20	11:10	2:50	170	8	9.5	1.5	0.88235294	0.2	Grey highly fractured rock	Greyish	4.3	2.15	1.1	1100	4	50
12:40	14:40	2:00	120	9.5	10	0.5	0.41666667	0.15	Grey highly fractured rock	Greyish	4.2	2.1	1.1	1100	4	50
7:10	9:40	2:30	150	10	11.5	1.5	1	0.07	Grey highly fractured rock	Greyish	3.9	2.73	1.1	1100	4	30
11:20	12:50	1:30	90	11.5	13	1.5	1.66666667	0.4	Grey highly fractured rock	Greyish	3.9	2.73	1.1	1100	4	30
14:10	16:40	2:30	150	13	14	1	0.66666667	0.9	Grey highly fractured rock	Greyish	3.85	2.7	1.1	1100	4	29.87013
17:30	19:10	1:40	100	14	15	1	1	0.3	Grey highly fractured rock	Greyish	3.6	2.52	1.1	1100	4	30
12:20	14:50	2:30	150	15	16.5	1.5	1	0.36	Grey highly fractured rock	Greyish	3.76	2.63	1.1	1100	4	30.05319
15:40	18:10	2:30	150	16.5	18	1.5	1	0.16	Grey highly fractured rock	Greyish	3.4	2.38	1.1	1100	4	30
7:10	9:50	2:40	160	18	19.5	1.5	0.9375	0.44	Grey highly fractured rock	Greyish	3.2	2.4	1.2	1100	4	25
11:30	13:30	2:00	120	19.5	21	1.5	1.25	0.46	Grey highly fractured rock	Greyish	3.2	2.4	1.2	1100	4	25
15:10	17:30	2:20	140	21	22.5	1.5	1.07142857	0.335	Grey highly fractured rock	Greyish	3.2	2.33	1.2	1100	4	27.1875
7:00	10:20	3:20	200	22.5	24	1.5	0.75	0.32	Grey highly fractured rock	Greyish	2.8	2.24	1.2	1100	4	20
13:20	17:40	4:20	260	24	25.5	1.5	0.57692308	0.81	Grey highly fractured rock	Greyish	2.75	2.2	1.2	1100	4	20
7:20	9:10	1:50	110	25.5	27	1.5	1.36363636	0.52	Grey highly fractured rock with clay	Greyish	2.68	2.14	1.25	1100	4	20.14925
11:00	13:40	2:40	160	27	28.5	1.5	0.9375	0.225	Grey highly fractured rock with clay	Greyish	2.4	1.92	1.3	1100	4	20
15:10	17:30	2:20	140	28.5	30	1.5	1.07142857	0.345	Grey highly fractured rock	Greyish	2.4	1.92	1.24	1100	4	20
7:00	8:40	1:40	100	30	31.5	1.5	1.5	0.245	Grey highly fractured rock	Greyish	2.1	1.68	1.25	1100	4	20
9:10	12:40	3:30	210	31.5	33	1.5	0.71428571	0.285	Grey highly fractured rock	Greyish	2.1	1.68	1.3	1100	4	20
14:20	16:50	2:30	150	33	34.5	1.5	1	0.255	Grey highly fractured rock	Greyish	2	1.6	1.24	1100	4	20
17:20	19:10	1:50	110	34.5	36	1.5	1.36363636	0.23	Grey highly fractured rock	Greyish	2	1.6	1.24	1100	4	20
7:30	10:10	2:40	160	36	37	1	0.625	0.18	Grey highly fractured rock	Greyish	1.98	1.68	1.48	1100	4	15.15152
10:50	12:20	1:30	90	37	37.5	0.5	0.55555556	0.07	Grey highly fractured rock	Greyish	2.02	1.72	1.5	1100	4	14.85149
7:00	9:45	2:45	165	37.5	39	1.5	0.90909091	0.06	Grey highly fractured rock	Greyish	2.05	1.74	1.67	1100	4	15.12195
10:20	13:10	2:50	170	39	40.5	1.5	0.88235294	0.11	Grey highly fractured rock	Greyish	1.9	1.62	1.7	1100	4	14.73684
13:50	16:20	2:30	150	40.5	42	1.5	1	0.18	Grey highly fractured rock	Greyish	1.95	1.66	1.75	1100	4	14.87179
16:35	18:10	1:35	95	42	43.5	1.5	1.57894737	0.14	Grey highly fractured rock	Greyish	1.96	1.67	1.76	1100	4	14.79592
7:30	10:15	2:45	165	43.5	45	1.5	0.90909091	0.245	Grey highly fractured rock	Greyish	1.8	1.53	1.85	1100	4	15
11:00	13:30	2:30	150	45	46.5	1.5	1	0.21	Grey highly fractured rock	Greyish	1.7	1.45	2.1	1100	4	14.70588
14:40	16:50	2:10	130	46.5	48	1.5	1.15384615	0.255	Grey highly fractured rock	Greyish	1.75	1.49	2.05	1100	4	14.85714
17:25	18:40	1:15	75	48	48.5	0.5	0.66666667	0.1	Grey highly fractured rock	Greyish	1.75	1.49	2.25	1100	4	14.85714
7:00	10:15	3:15	195	48.5	50	1.5	0.76923077	0.3	Grey highly fractured rock	Greyish	1.75	1.49	1.9	1100	4	14.85714
							0.9875548	0.27243								36.8702

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-18.0 m and deeper, the goundwater levels before drilling were observed as stable. In addition, leackage of drilling water was improved after the drilling of GL-18.0 m and deeper as well. Thus at the depth of GL-18.0 m and deeper, it is highly likely that less loosened bedrock lies. The depths of slip surface was, threfore, set at GL-18.0 m.

The stable groundwater level before drilling suggests that groundwater does not flow out from the fissures below GL-18.0 m. Thus the stable groundwater level at GL-7.0 m can be considered as the head of non-pressured groundwater stored in the stable bedrock.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 3 Line-2

Date: ..... from 12/4/2021 to 19/4/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
7:20	9:40	2:20	140	0	1.5	1.5	1.07142857	nil	Brownish Silty sand with gravel	Greyish	4.64	0	0	1100	4	100
10:10	13:10	3:00	180	1.5	3	1.5	0.83333333	nil	Brownish Silty sand with gravel	Greyish	5.06	0	0	1100	4	100
13:40	15:30	1:50	110	3	4	1	0.90909091	nil	Brownish Silty sand with gravel	Greyish	7.91	0	0	1100	4	100
16:00	17:20	1:20	80	4	5.5	1.5	1.875	nil	Brownish Silty sand with gravel	Greyish	16.75	0	0	1100	4	100
17:40	19:00	1:20	80	5.5	6.5	1	1.25	nil	Brownish Silty sand with gravel	Greyish	16.75	0	0	1100	4	100
6:40	8:50	2:10	130	6.5	7	0.5	0.38461538	0.05	Light Grey, Highly Fractured Rock	Greyish	5	0	0	1100	4	100
9:20	11:30	2:10	130	7	8	1	0.76923077	0.038	Light Grey, Highly Fractured Rock	Greyish	7	0	0	1100	4	100
12:00	14:00	2:00	120	8	9	1	0.83333333	nil	Light Grey, Highly Fractured Rock	Greyish	7.25	0	0	1100	4	100
15:00	17:30	2:30	150	9	10.5	1.5	1	nil	Light Grey, Highly Fractured Rock	Greyish	8.93	0	0	1100	4	100
7:15	10:50	3:35	215	10.5	11.5	1	0.46511628	nil	Light Grey, Highly Fractured Rock	Greyish	3.02	0	0	1100	4	100
11:20	13:00	1:40	100	11.5	12	0.5	0.5	nil	Light Grey, Highly Fractured Rock	Greyish	9.1	0	0	1100	4	100
14:00	16:40	2:40	160	12	13	1	0.625	nil	Light Grey, Highly Fractured Rock	Greyish	5.44	0	0	1100	4	100
17:00	18:20	1:20	80	13	13.5	0.5	0.625	nil	Light Grey, Highly Fractured Rock	Greyish	16.75	0	0	1100	4	100
7:40	9:30	1:50	110	13.5	14.5	1	0.90909091	0.04	Light Grey, Highly Fractured Rock	Greyish	5.91	3.55	2	1100	4	39.93232
10:10	12:30	2:20	140	14.5	15	0.5	0.35714286	0.03	Light Grey, Highly Fractured Rock	Greyish	6.5	3.9	2	1100	4	40
13:30	16:00	2:30	150	15	16	1	0.66666667	nil	Light Grey, Highly Fractured Rock	Greyish	5.8	3.48	2	1100	4	40
16:30	18:00	1:30	90	16	16.5	0.5	0.55555556	nil	Light Grey, Highly Fractured Rock	Greyish	14.89	8.93	2	1100	4	40.02686
7:20	10:10	2:50	170	16.5	17.5	1	0.58823529	nil	Light Grey, Highly Fractured Rock	Greyish	3.82	2.87	2.3	1100	4	24.86911
10:40	11:20	0:40	40	17.5	18	0.5	1.25	nil	Light Grey, Highly Fractured Rock	Greyish	22.75	17.06	2.3	1100	4	25.01099
7:00	11:30	4:30	270	18	19.5	1.5	0.55555556	0.07	Light Grey, Highly Fractured Rock	Greyish	2.41	2.41	3.5	1100	4	
12:10	13:00	0:50	50	19.5	20	0.5	1	0.11	Light Grey, Highly Fractured Rock	Greyish	18.2	18.2	3.5	1100	4	
14:00	16:30	2:30	150	20	21	1	0.66666667	0.07	Light Grey, Highly Fractured Rock	Greyish	5.8	5.8	3.5	900	6	
17:00	18:40	1:40	100	21	22	1	1	0.15	Light Grey, Highly Fractured Rock	Greyish	13.4	13.4	3.5	900	6	
6:40	8:40	2:00	120	22	23	1	0.83333333	0.08	Light Grey, Highly Fractured Rock	Greyish	5.42	5.42	3.5	900	6	
9:10	11:00	1:50	110	23	24	1	0.90909091	0.32	Light Grey, Highly Fractured Rock	Greyish	8.27	8.27	3.45	900	6	
11:30	13:30	2:00	120	24	25	1	0.83333333	0.114	Light Grey, Highly Fractured Rock	Greyish	7.25	7.25	3.5	1100	4	
14:20	16:40	2:20	140	25	26.5	1.5	1.07142857	0.06	Light Grey, Highly Fractured Rock	Greyish	9.57	9.57	3.4	1100	4	
17:10	18:50	1:40	100	26.5	27	0.5	0.5	0.025	Light Grey, Highly Fractured Rock	Greyish	13.4	13.4	3.4	1100	4	
7:20	9:10	1:50	110	27	28	1	0.90909091	0.08	Light Grey, Highly Fractured Rock	Greyish	5.91	5.91	3.5	1100	4	
9:50	11:20	1:30	90	28	28.5	0.5	0.55555556	0.06	Light Grey, Highly Fractured Rock	Greyish	10.11	10.11	3.5	1100	4	
11:50	13:00	1:10	70	28.5	29	0.5	0.71428571	0.12	Light Grey, Highly Fractured Rock	Greyish	12.43	12.43	3.5	1100	4	
14:00	16:30	2:30	150	29	30	1	0.66666667	0.13	Light Grey, Highly Fractured Rock	Greyish	8.93	8.93	3.56	1100	4	
							0.802589	0.091								47.1825

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-13.5 m and deeper, the goundwater levels before drilling were observed as stabl. In addition, leackage of drilling water was improved after the drilling of GL-13.5 m and deeper as well. Thus at the depth of GL-15.5 m and deeper, it is likely that less loosened bedrock lies.

On the other hand, at BV-4, slip surface wa sset at much deeper depths. Thus, considering the shape of slope surface, the depths of slip surface was set at GL-28.74 m.

The highest groundwater level during the groundwater monitoring was observed at the ground surface with minus value measurement. Thus GL-0.00 m was set as HWL.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 4 Line-2

Date: ..... from 31/1/2021 to 18/2/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered /Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
12:10	13:00	0:50	50	0	1	1	2	nil	Silty sand with gravel & pebble	Black	20	0	0	1100	4	100
13:15	14:30	1:15	75	1	1.5	0.5	0.66666667	nil	Silty sand with gravel & pebble	Black	6.67	0	0	1100	4	100
14:30	15:30	1:00	60	1.5	2	0.5	0.83333333	nil	Silty sand with gravel & pebble	Black	25	0	0	1100	4	100
15:30	16:30	1:00	60	2	2.5	0.5	0.83333333	nil	Silty sand with gravel & pebble	Black	25	0	0	1100	4	100
16:30	17:30	1:00	60	2.5	3	0.5	0.83333333	nil	Silty sand with gravel & pebble	Black	25	0	0	1100	4	100
8:00	10:00	2:00	120	3	3.5	0.5	0.41666667	nil	Silty sand with gravel & pebble	Black	8.33	0	0	1100	4	100
10:00	13:00	3:00	180	3.5	4.5	1	0.55555556	nil	Silty sand with gravel & pebble	Black	2.78	0	0	1100	4	100
13:00	15:30	2:30	150	4.5	5.5	1	0.66666667	nil	Silty sand with gravel & pebble	Black	10	0	0	1100	4	100
15:30	17:30	2:00	120	5.5	6.5	1	0.83333333	nil	Silty sand with gravel & pebble	Black	12.5	0	0	1100	4	100
17:30	19:00	1:30	90	6.5	7	0.5	0.55555556	nil	Silty sand with gravel & pebble	Black	16.67	0	0	1100	4	100
7:00	8:30	1:30	90	7	8	1	1.11111111	0.05	Highly fractured rock, light gray	Black	13.33	0	0	1000	4	100
8:30	10:00	1:30	90	8	9	1	1.11111111	0.063	Highly fractured rock, light gray	Black	16.67	0	0	1000	4	100
11:30	13:00	1:30	90	9	9.5	0.5	0.55555556	0.115	Highly fractured rock, light gray	Black	27.78	0	0	1000	4	100
13:00	18:00	5:00	300	9.5	10.5	1	0.33333333	0.15	Highly fractured rock, light gray	Black	16.67	0	0	1000	4	100
7:00	8:30	1:30	90	10.5	12	1.5	1.66666667	0.4	Highly fractured rock, light gray	Gray	13.33	0	0	1000	4	100
9:00	12:00	3:00	180	12	15	3	1.66666667	0.078	Sandstone	Gray	6.67	0	0	1000	4	100
13:00	18:30	5:30	330	15	18	3	0.90909091	0.186	Highly fractured rock	Gray	9.09	0	0	1000	4	100
8:00	10:30	2:30	150	18	19	1	0.66666667	0.115	Highly fractured rock	Gray	13.33	0	0	1000	4	100
11:00	13:00	2:00	120	19	19.5	0.5	0.41666667	0.12	Highly fractured rock	Gray	10	0	0	1000	4	100
14:00	18:00	4:00	240	19.5	21	1.5	0.625	0.098	Highly fractured rock	Gray	12.5	0	0	1000	4	100
12:30	14:00	1:30	90	21	21.5	0.5	0.55555556	0.04	Highly fractured rock	Gray	15.56	0	0	1000	4	100
14:30	16:00	1:30	90	21.5	22	0.5	0.55555556	0.035	Highly fractured rock	Gray	15.56	0	0	1000	4	100
16:30	18:30	2:00	120	22	22.5	0.5	0.41666667	0.061	Highly fractured rock	Gray	11.67	0	0	1000	4	100
8:00	10:00	2:00	120	22.5	23	0.5	0.41666667	0.251	Highly fractured rock	Gray	16.67	0	0	1000	4	100
10:00	12:30	2:30	150	23	23	0	0	nil	Highly fractured rock	Gray	13.33	0	0	1000	4	100
14:00	17:00	3:00	180	23	23.5	0.5	0.27777778	0.115	Highly fractured rock	Gray	11.11	0	0	1000	4	100
8:00	10:00	2:00	120	23.5	24	0.5	0.41666667	0.11	Highly fractured rock	Gray	16.67	0	0	1000	4	100
10:00	13:30	3:30	210	24	24.5	0.5	0.23809524	0.135	Highly fractured rock	Gray	13.33	0	0	1000	4	100
14:00	17:30	3:30	210	24.5	25	0.5	0.23809524	0.086	Highly fractured rock	Gray	16.67	0	0	1000	4	100
7:00	10:00	3:00	180	25	26	1	0.55555556	0.065	Highly fractured rock	Gray	11.11	0	0	1000	4	100
10:30	12:30	2:00	120	26	27.5	1.5	1.25	0.077	Highly fractured rock	Gray	16.67	0	0	1000	4	100
13:30	15:00	1:30	90	27.5	28	0.5	0.55555556	0.062	Highly fractured rock	Gray	27.78	0	0	1000	4	100
15:30	18:00	2:30	150	28	28.5	0.5	0.33333333	0.171	Highly fractured rock	Gray	13.33	0	0	1000	4	100
7:00	9:30	2:30	150	28.5	29.5	1	0.66666667	nil	Highly fractured rock	Gray	13.33	0	0	1000	4	100
10:00	13:00	3:00	180	29.5	30.5	1	0.55555556	nil	Highly fractured rock	Gray	11.11	0	0	1000	4	100
14:00	16:00	2:00	120	30.5	32	1.5	1.25	0.114	Highly fractured rock	Gray	13.89	0	0	1000	4	100
16:30	18:00	1:30	90	32	32.5	0.5	0.55555556	nil	Highly fractured rock	Gray	16.67	0	0	1000	4	100
7:00	10:00	3:00	180	32.5	33.5	1	0.55555556	0.112	Highly fractured rock	Gray	11.11	3.33	0	1000	4	70.027
11:00	13:30	2:30	150	33.5	34	0.5	0.33333333	nil	Highly fractured rock	Gray	13.33	4	0	1000	4	69.9925
14:00	16:00	2:00	120	34	35	1	0.83333333	nil	Highly fractured rock	Gray	16.67	5	0	1000	4	70.006
16:30	18:30	2:00	120	35	36	1	0.83333333	nil	Highly fractured rock	Gray	12.5	3.75	0	1000	4	70
7:00	9:00	2:00	120	36	37	1	0.83333333	nil	Highly fractured rock	Gray	12.5	5.63	0	1000	4	54.96
9:30	12:30	3:00	180	37	37.5	0.5	0.27777778	0.15	Highly fractured rock	Gray	8.33	3.75	0	1000	4	54.98199
13:30	17:00	3:30	210	37.5	39	1.5	0.71428571	0.84	Highly fractured rock	Gray	9.52	4.29	0	1000	4	54.93697
17:30	19:30	2:00	120	39	40.5	1.5	1.25	nil	Highly fractured rock	Gray	13.33	6	0	1000	4	54.98875
7:00	10:20	3:20	200	40.5	42	1.5	0.75	0.078	Highly fractured rock	Gray	12.5	6.25	0	1000	4	50
11:00	13:30	2:30	150	42	44.5	2.5	1.66666667	0.179	Highly fractured rock	Gray	8.33	4.17	0	1000	4	49.93998
14:00	16:30	2:30	150	44.5	46.5	2	1.33333333	nil	Highly fractured rock	Gray	9.52	4.76	0	1000	4	50
7:00	10:00	3:00	180	46.5	48	1.5	0.83333333	nil	Highly fractured rock	Gray	13.33	6.67	0	1000	4	49.96249
10:30	12:30	2:00	120	48	50	2	1.66666667	nil	Highly fractured rock	Gray	13.33	6.67	0	1000	4	49.96249
							0.7594913	0.14468								88.9952

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-23.5 m and deeper, the goundwater levels before drilling were observed as stable. On the other hand, leackage of drilling water was improved at 2 depths of GL-32.5 m and GL-36.0 m.

According to the landslide monitoring, a large displacement was observed at GL-36.0m on the 1st of October, 2021.

The depths of slip surface was, threfore, set at GL-36.0 m.

The stable groundwater level before drilling suggests that groundwater does not flow out from the fissures below GL-36.0 m. Thus the stable groundwater level at GL-13.0 m can be considered as the head of non-pressured groundwater stored in the stable bedrock.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 5 Line-2

Date: ..... from 5/5/2021 to 25/5/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
11:00	13:00	2:00	120	0	1	1	0.83333333	0.09	Brownish Silty sand with gravel	Greyish	10.83	0	0	1100	4	100
14:00	16:30	2:30	150	1	2	1	0.66666667	0.08	Brownish Silty sand with gravel	Greyish	11	0	0	1100	4	100
8:00	11:00	3:00	180	2	3	1	0.55555556	0.13	Light Grey, Highly Fractured Rock	Greyish	7.78	0	0	1100	4	100
11:30	13:10	1:40	100	3	3.5	0.5	0.5	0.05	Light Grey, Highly Fractured Rock	Greyish	8.1	0	0	1100	4	100
14:00	16:40	2:40	160	3.5	4.5	1	0.625	0.04	Light Grey, Highly Fractured Rock	Greyish	8.75	0	0	1100	4	100
17:20	18:30	1:10	70	4.5	5	0.5	0.71428571	0.03	Light Grey, Highly Fractured Rock	Greyish	13.43	0	0	1000	6	100
12:00	13:20	1:20	80	5	5.5	0.5	0.625	0.18	Light Grey, Highly Fractured Rock	Greyish	12.25	0	0	1000	6	100
14:00	15:30	1:30	90	5.5	6.5	1	1.11111111	0.1	Light Grey, Highly Fractured Rock	Greyish	15.78	0	0	1000	6	100
16:00	17:40	1:40	100	6.5	7	0.5	0.5	0.12	Light Grey, Highly Fractured Rock	Greyish	13.5	0	0	1000	6	100
7:00	10:10	3:10	190	7	8.5	1.5	0.78947368	0.5	Light Grey, Highly Fractured Rock	Greyish	11.32	0	0	1000	6	100
10:50	13:00	2:10	130	8.5	9.5	1	0.76923077	0.25	Light Grey, Highly Fractured Rock	Greyish	9.31	0	0	1000	6	100
14:30	16:20	1:50	110	9.5	10	0.5	0.45454545	0.17	Light Grey, Highly Fractured Rock	Greyish	7.91	0	0	1000	6	100
16:50	18:00	1:10	70	10	10.5	0.5	0.71428571	0.14	Light Grey, Highly Fractured Rock	Greyish	13.43	0	0	1000	6	100
9:10	11:30	2:20	140	10.5	11.5	1	0.71428571	0.3	Light Grey, Highly Fractured Rock	Greyish	9.64	0	0	1000	6	100
12:10	13:30	1:20	80	11.5	12.5	1	1.25	0.32	Light Grey, Highly Fractured Rock	Greyish	11.38	0	0	1000	6	100
14:30	16:30	2:00	120	12.5	14	1.5	1.25	0.55	Light Grey, Highly Fractured Rock	Greyish	17.25	0	0	1000	6	100
17:00	18:30	1:30	90	14	15	1	1.11111111	0.25	Light Grey, Highly Fractured Rock	Greyish	14.89	0	0	1000	6	100
11:30	13:10	1:40	100	15	16	1	1	0.71	Light Grey, Highly Fractured Rock	Greyish	14.5	0	0	1000	6	100
14:20	17:00	2:40	160	16	17.5	1.5	0.9375	0.95	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	13.13	0	0	1000	6	100
13:00	16:30	3:30	210	17.5	18.5	1	0.47619048	0.31	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	7.86	0	0	1000	6	100
7:10	9:40	2:30	150	18.5	19.5	1	0.66666667	0.32	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	9.67	0	0	900	6	100
10:10	13:00	2:50	170	19.5	21	1.5	0.88235294	0.58	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	12.41	0	0	900	6	100
14:00	16:10	2:10	130	21	22	1	0.76923077	0.35	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	10.54	0	0	900	6	100
16:50	18:20	1:30	90	22	22.5	0.5	0.55555556	0.21	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	14.89	0	0	900	6	100
14:30	15:50	1:20	80	22.5	23.5	1	1.25	0.21	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	18.13	0	0	900	6	100
16:20	17:40	1:20	80	23.5	24.5	1	1.25	0.17	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	13.1	0	0	900	6	100
9:00	13:00	4:00	240	24.5	26	1.5	0.625	0.42	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	11.46	0	0	900	6	100
14:00	16:40	2:40	160	26	27	1	0.625	0.2	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	7.56	0	0	900	6	100
17:10	18:30	1:20	80	27	27.5	0.5	0.625	0.14	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	10.88	0	0	900	6	100
9:00	12:00	3:00	180	27.5	28.5	1	0.55555556	0.545	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	8.06	0	0	900	6	100
7:00	8:40	1:40	100	28.5	29	0.5	0.5	0.05	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	9.5	0	0	900	6	100
9:10	13:10	4:00	240	29	30	1	0.41666667	0.115	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	6.71	0	0	900	6	100
14:00	17:00	3:00	180	30	31	1	0.55555556	0.095	Silty clay with gravel	Greyish	9.83	0	0	900	6	100
17:25	19:10	1:45	105	31	31.5	0.5	0.47619048	0.02	Silty clay with gravel	Greyish	8	0	0	900	6	100
8:00	10:30	2:30	150	31.5	32.5	1	0.66666667	0.075	Silty clay with gravel	Greyish	9.67	0	0	900	6	100
11:00	14:00	3:00	180	32.5	34	1.5	0.83333333	0.11	Light Grey, Highly Fractured Rock	Greyish	11.17	0	0	900	6	100
15:30	16:50	1:20	80	34	35	1	1.25	0.06	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	14.63	0	0	900	6	100
17:10	18:30	1:20	80	35	36	1	1.25	0.092	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	16.75	0	0	900	6	100
8:00	10:10	2:10	130	36	37.5	1.5	1.15384615	0.125	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	15.42	0	0	900	6	100
10:40	14:00	3:20	200	37.5	39	1.5	0.75	0.13	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	9.3	0	0	900	6	100
15:30	17:40	2:10	130	39	40.5	1.5	1.15384615	0.12	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	15.15	0	0	900	6	100
18:10	19:30	1:20	80	40.5	42	1.5	1.875	0.125	Dark Grey, Highly weathered Rock with small pieces of core	Greyish	16.75	0	0	900	6	100
7:00	9:00	2:00	120	42	43	1	0.83333333	0.17	Dark Grey, Highly weathered Rock with small pieces of core	Greyish	11.25	0	0	900	6	100
9:40	11:40	2:00	120	43	44.5	1.5	1.25	0.32	Dark Grey, Highly weathered Rock with small pieces of core	Greyish	15.92	0	0	900	6	100
12:10	13:30	1:20	80	44.5	45.5	1	1.25	0.19	Dark Grey, Highly weathered Rock with small pieces of core	Greyish	15.88	0	0	900	6	100
14:20	16:30	2:10	130	45.5	47	1.5	1.15384615	0.345	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	14.92	0	0	900	6	100
17:20	19:00	1:40	100	47	48	1	1	0.14	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	13.4	0	0	900	6	100
6:30	8:00	1:30	90	48	49	1	1.11111111	0.08	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	15	0	0	900	6	100
8:40	11:10	2:30	150	49	50	1	0.66666667	0.11	Dark Grey, Highly Fractured Rock with small pieces of core	Greyish	9.4	0	0	900	6	100
							0.8479183	0.22218								100

Interpretation and basis for design

No water table was formed during drilling and groundwater level variation analysis was not possible.

At the depth of GL-36.0 m and deeper, however, casings were not installed along drilling work. It is likely that at the sectio deeper than GL-36.0m, collapse of borehole wall was light and thus bedrock is not so loosened.

Since all drilling water leacked from fissures, for all depth fo boring bedrock is somewaht loosened. The depths of slip surface was, threfore, set at the botom of the drilling of GL-50.0 m.

The highest groundwater level during the groundwater monitoring was observed at GL-30.0 m on the 17th of June, 2021. Thus GL-30.0 m was set as HWL.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 6 Line-2

Date: ..... from 23/6/2021 to 13/7/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
7:00	9:20	2:20	140	0	1	1	0.71428571	nil	Soil	Greyish	5.71	0	0	1100	4	100
10:00	13:10	3:10	190	1	1.5	0.5	0.26315789	nil	Soil	Greyish	6.32	0	0	1100	4	100
14:00	17:10	3:10	190	1.5	2.5	1	0.52631579	nil	Boulder with silty clay	Greyish	4.21	0	0	1100	4	100
17:40	19:00	1:20	80	2.5	3	0.5	0.625	nil	Boulder with silty clay	Greyish	5	0	0	1100	4	100
7:10	8:20	1:10	70	3	3.5	0.5	0.71428571	nil	Boulder with silty clay	Greyish	5.71	0	0	1100	4	100
8:50	12:30	3:40	220	3.5	4.5	1	0.45454545	nil	Boulder with silty clay	Greyish	3.64	0	0	1100	4	100
13:30	16:40	3:10	190	4.5	5.5	1	0.52631579	nil	Boulder with silty clay	Greyish	4.21	0	0	1100	4	100
17:30	18:40	1:10	70	5.5	6	0.5	0.71428571	nil	Boulder with silty clay	Greyish	5.71	0	0	1100	4	100
7:30	8:40	1:10	70	6	6.25	0.25	0.35714286	nil	Boulder with silty clay	Greyish	2.86	0	0	1100	4	100
9:10	11:30	2:20	140	6.25	6.75	0.5	0.35714286	nil	Boulder with silty clay	Greyish	2.86	0	0	1100	4	100
12:00	14:10	2:10	130	6.75	7.75	1	0.76923077	nil	Boulder with silty clay	Greyish	6.15	0	0	1100	4	100
15:00	16:20	1:20	80	7.75	8.5	0.75	0.9375	nil	Light Grey, Highly Fractured Rock with clay	Greyish	11.25	0	0	1100	4	100
17:00	19:00	2:00	120	8.5	9	0.5	0.41666667	nil	Light Grey, Highly Fractured Rock with clay	Greyish	4.62	0	0	1100	4	100
7:00	10:30	3:30	210	9	10	1	0.47619048	nil	Light Grey, Highly Fractured Rock with clay	Greyish	5.71	0	0	1100	4	100
11:10	13:10	2:00	120	10	11	1	0.83333333	nil	Light Grey, Highly Fractured Rock with clay	Greyish	10	0	0	1100	4	100
14:00	17:00	3:00	180	11	11.75	0.75	0.41666667	nil	Light Grey, Highly Fractured Rock with clay	Greyish	5	0	0	1100	4	100
17:30	18:40	1:10	70	11.75	12	0.25	0.35714286	nil	Light Grey, Highly Fractured Rock with clay	Greyish	4.29	0	0	1100	4	100
10:10	11:40	1:30	90	12	12.5	0.5	0.55555556	nil	Light Grey, Highly Fractured Rock with clay	Greyish	6.67	0	0	1100	4	100
13:00	15:20	2:20	140	12.5	13	0.5	0.35714286	nil	Light Grey, Highly Fractured Rock with clay	Greyish	4.29	0	0	1100	4	100
16:10	18:20	2:10	130	13	13.5	0.5	0.38461538	nil	Light Grey, Highly Fractured Rock with clay	Greyish	4.62	0	0	1100	4	100
6:30	8:40	2:10	130	13.5	14.5	1	0.76923077	nil	Light Grey, Highly Fractured Rock with clay	Greyish	9.23	0	0	1100	4	100
10:00	13:00	3:00	180	14.5	16	1.5	0.83333333	nil	Light Grey, Highly Fractured Rock with clay	Greyish	10	0	0	1100	4	100
14:00	16:10	2:10	130	16	17	1	0.76923077	0.06	Light Grey, Highly Fractured Rock	Greyish	6.15	0	0	1000	6	100
17:30	19:10	1:40	100	17	18	1	1	0.045	Light Grey, Highly Fractured Rock	Greyish	8.1	0	0	1000	6	100
5:30	7:20	1:50	110	18	19.5	1.5	1.36363636	0.11	Light Grey, Highly Fractured Rock	Greyish	21.82	0	0	1000	6	100
9:30	11:30	2:40	160	19.5	21.5	2	1.25	0.5	Light Grey, weathered Rock	Greyish	20	0	0	1000	6	100
13:00	15:30	2:30	150	21.5	23	1.5	1	0.35	Light Grey, Highly Fractured Rock	Greyish	16	0	0	1000	6	100
16:40	19:00	2:20	140	23	25	2	1.42857143	0.3	Light Grey, Highly Fractured Rock	Greyish	22.86	0	0	1000	6	100
6:20	8:50	2:30	150	25	25.75	0.75	0.5	0.055	Light Grey, Highly Fractured Rock	Greyish	8	0	0	1000	6	100
9:30	12:30	3:00	180	25.75	27	1.25	0.69444444	0.045	Light Grey, Highly Fractured Rock	Greyish	11.11	0	0	1000	6	100
13:10	15:20	2:10	130	27	27.5	0.5	0.38461538	0.25	Light Grey, Highly Fractured Rock	Greyish	6.15	0	0	1000	6	100
16:20	19:20	3:00	180	27.5	29	1.5	0.83333333	0.7	Light Grey, Highly Fractured Rock	Greyish	13.33	0	0	1100	6	100
5:50	9:30	3:40	220	29	30	1	0.45454545	0.16	Light Grey, weathered Rock	Greyish	7.09	0	0	1000	6	100

10:20	13:30	3:10	190	30	31.5	1.5	0.78947368	0.095	Light Grey, weathered Rock	Greyish	12.63	0	0	1000	6	100
14:30	15:40	1:10	70	31.5	32.5	1	1.42857143	0.08	Light Grey, weathered Rock	Greyish	22.86	0	0	1000	6	100
16:20	18:50	2:30	150	32.5	34	1.5	1	0.3	Light Grey, weathered Rock	Greyish	16	0	0	1000	6	100
19:30	22:00	2:30	150	34	36	2	1.33333333	0.2	Light Grey, Highly Fractured Rock	Greyish	21.33	0	0	1000	6	100
6:30	8:30	2:00	120	36	36.5	0.5	0.41666667	0.12	Light Grey, weathered Rock	Greyish	10	0	0	1000	6	100
10:50	13:00	2:10	130	36.5	37.25	0.75	0.57692308	0.15	Light Grey, weathered Rock	Greyish	11.15	0	0	1000	6	100
14:40	16:20	1:40	100	37.25	37.5	0.25	0.25	0.08	Light Grey, weathered Rock	Greyish	9	0	0	1000	6	100
17:00	18:40	1:40	100	37.5	38	0.5	0.5	0.07	Light Grey, weathered Rock	Greyish	12	0	0	1000	6	100
5:00	7:40	2:40	160	38	39.5	1.5	0.9375	0.11	Light Grey, weathered Rock	Greyish	11.56	4.63	0	1000	6	59.9481
8:20	11:20	3:00	180	39.5	40.5	1	0.55555556	0.08	Light Grey, weathered Rock	Greyish	9.72	3.89	0	1000	6	59.97942
11:50	14:20	2:30	150	40.5	42	1.5	1	0.12	Light Grey, weathered Rock	Greyish	11.67	4.67	1.1	1000	6	59.98286
15:10	17:20	2:10	130	42	43.5	1.5	1.15384615	0.134	Light Grey, weathered Rock	Greyish	3.46	1.38	1.2	1000	6	60.11561
17:45	18:50	1:05	65	43.5	45	1.5	2.30769231	0.07	Light Grey, Highly Fractured Rock	Greyish	3.23	1.29	1.2	1000	6	60.06192
19:15	20:25	1:10	70	45	46	1	1.42857143	0.06	Light Grey, Highly Fractured Rock	Greyish	4	1.6	1.5	1000	6	60
7:00	9:40	2:40	160	46	47.5	1.5	0.9375	0.09	Light Grey, weathered Rock	Greyish	2	1.3	1.2	1000	6	35
10:10	12:50	2:40	160	47.5	48.5	1	0.625	0.07	Light Grey, weathered Rock	Greyish	1.44	0.93	1.2	1000	6	35.41667
13:00	15:10	2:10	130	48.5	50	1.5	1.15384615	0.06	Light Grey, weathered Rock	Greyish	2.31	1.96	1.2	1000	6	15.15152
11:20	13:10	1:50	110	50	51	1	0.90909091	0.11	Light Grey, weathered Rock	Greyish	3.09	2.41	1.2	1000	6	22.00647
14:00	16:10	2:10	130	51	52.5	1.5	1.15384615	0.1	Light Grey, weathered Rock	Greyish	1.69	1.32	1.2	1000	6	21.89349
16:40	17:30	0:50	50	52.5	53	0.5	1	0.12	Light Grey, weathered Rock	Greyish	3.6	2.81	1.2	1000	6	21.94444
7:00	9:40	2:40	160	53	54.5	1.5	0.9375	0.17	Light Grey, weathered Rock	Greyish	2	1.7	1.2	1000	6	15
10:10	11:20	1:10	70	54.5	55	0.5	0.71428571	0.11	Light Grey, weathered Rock	Greyish	3.29	2.79	1.2	1000	6	15.19757
							0.7844848	0.13861								84.3945

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-46.0 m and deeper, the goundwater levels before drilling were observed as stable. In addition, leackage of drilling water was improved after the drilling of GL-46.0 m and deeper as well. Thus at the depth of GL-46.0 m and deeper, it is highly likely that less loosened bedrock lies. The depths of slip surface was, threfore, set at GL-46.0 m.

The stable groundwater level before drilling suggests that groundwater does not flow out from the fissures below GL-46.0 m. Thus the stable groundwater level at GL-41.0 m can be considered as the head of non-pressured groundwater stored in the stable bedrock.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 7' Line-2

Date: ..... from 20/7/2021 to 28/7/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
5:30	8:20	2:50	170	0	1.5	1.5	0.88235294	nil	Silty clay	Greyish	1.47	0	0	1100	4	100
9:00	11:40	2:40	160	1.5	2.5	1	0.625	nil	Silty clay	Greyish	1.25	0	0	1100	4	100
2:30	15:10	2:40	160	2.5	4	1.5	0.9375	nil	Silty clay with boukder	Greyish	4.69	0	0	1100	4	100
6:00	18:20	2:20	140	4	5.5	1.5	1.07142857	nil	Silty clay with boukder	Greyish	7.14	0	0	1100	4	100
8:50	19:40	0:50	50	5.5	6	0.5	1	nil	Grey highly fractured rock with clay	Greyish	9	0	0	1100	4	100
5:00	7:40	2:40	160	6	7.5	1.5	0.9375	nil	Grey highly fractured rock with clay	Greyish	7.5	0	0	1100	4	100
8:20	11:00	2:40	160	7.5	9	1.5	0.9375	nil	Grey highly fractured rock with clay	Greyish	7.38	0	0	1100	4	100
1:50	14:10	2:20	140	9	10.5	1.5	1.07142857	nil	Grey highly fractured rock with clay	Greyish	9.36	0	0	1100	4	100
5:00	17:40	2:40	160	10.5	11.5	1	0.625	nil	Grey highly fractured rock with clay	Greyish	8.13	0	0	1100	4	100
8:10	19:15	1:05	65	11.5	12	0.5	0.76923077	nil	Grey highly fractured rock with clay	Greyish	8.62	0	0	1100	4	100
5:20	7:30	2:10	130	12	13.5	1.5	1.15384615	nil	Grey highly fractured rock with clay	Greyish	8.46	0	0	1100	4	100
8:00	10:30	2:30	150	13.5	14.5	1	0.66666667	nil	Grey highly fractured rock with clay	Greyish	7.89	0	0	1100	4	100
1:30	14:30	3:00	180	14.5	16	1.5	0.83333333	nil	Grey highly fractured rock with clay	Greyish	8.37	0	0	1100	4	100
5:10	17:20	2:10	130	16	17.5	1.5	1.15384615	nil	Grey highly fractured rock with clay	Greyish	6.25	1.25	0.9	1100	4	80
8:00	18:50	0:50	50	17.5	18	0.5	1	nil	Grey highly fractured rock with clay	Greyish	6.13	1.23	0.95	1100	4	79.93475
4:40	7:00	2:20	140	18	19	1	0.71428571	0.06	Grey highly fractured rock with clay	Greyish	6.41	3.85	1.1	1100	4	39.9376
7:20	9:40	2:20	140	19	20.5	1.5	1.07142857	0.05	Grey highly fractured rock with clay	Greyish	6.2	3.72	1.1	1100	4	40
0:00	12:30	2:30	150	20.5	22	1.5	1	0.12	Grey highly fractured rock	Greyish	5.98	3.59	1.1	1100	4	39.96656
3:10	13:30	0:20	20	22	23.5	1.5	7.5	0.1	Grey highly fractured rock	Greyish	5.87	3.52	1.1	1100	4	40.03407
4:20	16:40	2:20	140	23.5	24.5	1	0.71428571	0.15	Grey highly fractured rock	Greyish	5.68	3.41	1.1	1100	4	39.96479
7:10	19:10	2:00	120	24.5	25	0.5	0.41666667	0.03	Grey highly fractured rock	Greyish	5.74	3.44	1.1	1100	4	40.06969
7:10	9:50	2:40	160	25	26.5	1.5	0.9375	0.21	Grey highly fractured rock	Greyish	5.71	4.57	1.1	1100	4	19.96497
0:40	12:00	1:20	80	26.5	27.5	1	1.25	0.125	Grey highly fractured rock	Greyish	5.36	4.29	1.1	1100	4	19.96269
5:10	17:00	1:50	110	27.5	28.5	1	0.90909091	0.09	Grey highly fractured rock	Greyish	5.87	4.7	1.1	1100	4	19.93186
7:40	19:10	1:30	90	28.5	29	0.5	0.55555556	0.13	Grey highly fractured rock	Greyish	5.71	4.57	1.1	1100	4	19.96497
6:10	8:50	2:40	160	29	30.5	1.5	0.9375	0.08	Grey highly fractured rock	Greyish	5.12	4.1	1.2	1100	4	19.92188
9:30	11:40	2:10	130	30.5	31.5	1	0.76923077	0.12	Grey highly fractured rock	Greyish	4.87	3.9	1.2	1100	4	19.91786
6:10	17:40	1:30	90	31.5	32.5	1	1.11111111	0.09	Grey highly fractured rock	Greyish	4.98	3.98	1.2	1100	4	20.08032
8:30	19:20	0:50	50	32.5	33	0.5	1	0.04	Grey highly fractured rock	Greyish	4.78	3.82	1.2	1100	4	20.08368
4:10	15:50	1:40	100	33	34.5	1.5	1.5	0.06	Grey highly fractured rock	Greyish	4.12	3.3	1.2	1100	4	15.04854
6:20	18:50	2:30	150	34.5	36	1.5	1	0.05	Grey highly fractured rock	Greyish	3.98	3.38	1.2	1100	4	15.07538
5:20	8:10	2:50	170	36	37.5	1.5	0.88235294	0.17	Grey highly fractured rock with clay	Greyish	3.21	2.73	1.25	1100	4	14.95327
2:40	17:10	2:30	150	37.5	39	1.5	1	0.16	Grey highly fractured rock with clay	Greyish	3.48	2.96	1.3	1100	4	14.94253
3:00	15:40	2:40	160	39	40.5	1.5	0.9375	0.14	Grey highly fractured rock	Greyish	4.12	3.5	1.24	1100	4	15.04854
6:20	18:50	2:30	150	40.5	42	1.5	1	0.18	Grey highly fractured rock	Greyish	3.9	3.32	1.25	1100	4	14.87179
7:30	8:50	3:20	200	42	43.5	1.5	0.75	0.18	Grey highly fractured rock with clay	Greyish	3.68	3.13	1.25	1100	4	14.94565
9:20	11:40	2:20	140	43.5	44	0.5	0.35714286	0.14	Grey highly fractured rock with clay	Greyish	3.74	3.18	1.3	1100	4	14.97126
5:20	17:10	1:50	110	44	45	1	0.90909091	0.16	Grey highly fractured rock	Greyish	4.15	3.53	1.24	1100	4	14.93976

17:40	19:10	1:30	90	45	46	1	1.11111111	0.13	Grey highly fractured rock	Greyish	3.86	3.28	1.25	1100	4	15.02591
6:10	8:20	2:10	130	46	47.5	1.5	1.15384615	0.16	Grey highly fractured rock with clay	Greyish	3.87	3.48	1.25	1100	4	10.07757
9:00	12:20	3:20	200	47.5	49	1.5	0.75	0.21	Grey highly fractured rock with clay	Greyish	4.1	3.69	1.3	1100	4	10
14:20	16:40	2:20	140	49	50	1	0.71428571	0.23	Grey highly fractured rock	Greyish	3.79	3.41	1.24	1100	4	10.02639
							0.9052834	0.12463								48.5634

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-18.0 m and deeper, the goundwater levels before drilling were observed as stable. In addition, leackage of drilling water was improved after the drilling of GL-18.0 m and deeper as well. Thus at the depth of GL-18.0 m and deeper, it is highly likely that less loosened bedrock lies. The depths of slip surface was, threfore, set at GL-18.0 m.

The stable groundwater level before drilling suggests that groundwater does not flow out from the fissures below GL-18.0 m. Thus the stable groundwater level at GL-15.0 m can be considered as the head of non-pressured groundwater stored in the stable bedrock.

This bore hole of BV-7' was not on the main survey line of L2, however. The data of this borehole was not used for analysis.

Daily Drilling Operation Report

Project Name: CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NAKIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Borehole Number: ..... BV 10 Line-

Date: ..... from 11/1/2021 to 26/1/2021

Location: ..... 163+550, NH54

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	WATER PRESSURE [KpA]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	Leakage ratio [%]
From	To	hh:mm	minute	From	To	Meter drilled										
6:00	8:00	2:00	120	0	1	1	0.83333333	nil	Sandy gravel	Black	10	0	0	1000	4	100
8:30	12:00	3:30	210	1	1.5	0.5	0.23809524	nil	Sandy gravel	Black	4.29	0	0	1000	4	100
12:30	14:00	1:30	90	1.5	1.75	0.25	0.27777778	nil	Sandy gravel	Black	10	0	0	1000	4	100
15:00	17:30	2:30	150	1.75	2	0.25	0.16666667	nil	Sandy gravel	Black	11.11	0	0	1000	4	100
7:00	9:00	2:00	120	2	3	1	0.83333333	nil	Sandy gravel	Black	15	0	0	1000	4	100
9:30	12:00	2:30	150	3	4	1	0.66666667	nil	Sandy gravel	Black	13.33	0	0	1000	4	100
12:40	14:30	1:50	110	4	4.5	0.5	0.45454545	0.381	Grey shale rock	Black	12.75	0	0	1000	4	100
15:30	18:00	2:30	150	4.5	5	0.5	0.33333333	0.051	Grey shale rock	Black	12	0	0	1000	4	100
6:30	8:30	2:00	120	5	6	1	0.83333333	0.09	Grey shale rock	Black	20.83	0	0	900	6	100
9:00	11:00	2:00	120	6	6.5	0.5	0.41666667	0.381	Grey shale rock	Black	12.5	0	0	900	6	100
11:30	14:00	2:30	150	6.5	7	0.5	0.33333333	0.05	Grey shale rock	Black	18.67	0	0	1000	6	100
15:00	17:30	2:30	150	7	7.5	0.5	0.33333333	0.089	Grey shale rock	Black	18.67	0	0	1100	6	100
7:00	9:00	2:00	120	7.5	9	1.5	1.25	0.063	Grey shale rock	Greyish	29.17	0	0	1000	4	100
9:00	11:00	2:00	120	9	10	1	0.83333333	nil	Grey shale rock	Greyish	20.83	0	0	1000	4	100
11:30	14:00	2:30	150	10	10.5	0.5	0.33333333	0.05	Grey shale rock	Greyish	16.67	0	0	1000	4	100
15:00	17:30	2:30	150	10.5	11.5	1	0.66666667	0.038	Grey shale rock	Greyish	26.67	0	0	1000	4	100
8:00	10:00	2:00	120	11.5	13	1.5	1.25	0.038	Grey shale rock	Greyish	33.33	0	0	1000	4	100
10:30	12:30	2:00	120	13	14	1	0.83333333	0.05	Grey shale rock	Greyish	29.17	0	0	1000	4	100
13:00	14:00	1:00	60	14	14.5	0.5	0.83333333	0.355	Grey shale rock	Greyish	58.33	0	0	1000	6	100
15:00	18:00	3:00	180	14.5	16	1.5	0.83333333	0.05	Grey shale rock	Greyish	27.78	0	0	1000	6	100
8:00	10:00	2:00	120	16	17	1	0.83333333	0.038	Grey shale rock	Greyish	25	0	0	1000	4	100
10:30	12:30	2:00	120	17	18	1	0.83333333	0.05	Grey shale rock	Greyish	29.17	0	0	1000	4	100
13:00	14:00	1:00	60	18	18.5	0.5	0.83333333	0.038	Grey shale rock	Greyish	41.67	0	0	1000	4	100
15:00	18:00	3:00	180	18.5	19	0.5	0.27777778	0.038	Grey shale rock	Greyish	16.67	0	0	1000	4	100
7:00	9:00	2:00	120	19	20	1	0.83333333	0.04	Grey shale rock	Greyish	20	0	0	1000	4	100
9:30	11:30	2:00	120	20	20.5	0.5	0.41666667	0.05	Grey shale rock	Greyish	25	0	0	1000	4	100
12:00	14:00	2:00	120	20.5	21	0.5	0.41666667	0.038	Grey shale rock	Greyish	29.17	0	0	800	8	100
15:00	18:00	3:00	180	21	22	1	0.55555556	0.005	Grey shale rock	Greyish	22.22	0	0	800	8	100
7:00	9:00	2:00	120	22	23	1	0.83333333	0.05	Grey shale rock	Greyish	20.83	0	0	1000	4	100
9:30	11:30	2:00	120	23	23.5	0.5	0.41666667	0.03	Grey shale rock	Greyish	25	0	0	1000	4	100
12:00	14:00	2:00	120	23.5	24	0.5	0.41666667	0.04	Grey shale rock	Greyish	25	0	0	1000	4	100
15:00	18:00	3:00	180	24	25	1	0.55555556	0.03	Grey shale rock	Greyish	16.67	0	0	1000	4	100
8:00	10:00	2:00	120	25	26	1	0.83333333	0.05	Grey shale rock	Greyish	12.5	0	0	1000	4	100
10:30	12:30	2:00	120	26	27	1	0.83333333	0.04	Grey shale rock	Greyish	12.5	0	0	1000	4	100
13:00	15:00	2:00	120	27	28	1	0.83333333	0.076	Grey shale rock	Greyish	16.67	0	0	1000	4	100
15:30	18:00	2:30	150	28	28.5	0.5	0.33333333	0.07	Grey shale rock	Greyish	6.67	0	0	1000	4	100
8:00	10:00	2:00	120	28.5	30	1.5	1.25	0.635	Grey shale rock	Greyish	10	5	2	1000	6	50
10:30	12:30	2:00	120	30	31.5	1.5	1.25	0.584	Grey shale rock	Greyish	10	5	2	1000	6	50
13:00	15:00	2:00	120	31.5	33	1.5	1.25	0.685	Grey shale rock	Greyish	10	5	2	1000	6	50
15:30	18:00	2:30	150	33	34	1	0.66666667	0.584	Grey shale rock	Greyish	6.67	3.33	2	1000	6	50.07496
8:20	10:30	2:10	130	34	35.5	1.5	1.15384615	nil	Grey shale rock	Greyish	13.64	9.55	3	900	6	29.98534
11:00	13:10	2:10	130	35.5	37	1.5	1.15384615	nil	Grey shale rock	Greyish	13.08	9.15	3	900	6	30.04587
13:30	15:10	1:40	100	37	38	1	1	nil	Grey shale rock	Greyish	6	4.2	3	900	6	30
15:40	18:10	2:30	150	38	39	1	0.66666667	nil	Grey shale rock	Greyish	5.33	3.73	3	900	6	30.01876
8:10	10:00	1:50	110	39	40	1	0.90909091	nil	Grey shale rock	Greyish	6.36	4.77	3	1100	8	25
10:30	12:15	1:45	105	40	41	1	0.95238095	0.254	Grey shale rock	Greyish	5.38	4.04	4	1100	8	24.90706
12:50	18:00	5:10	310	41	43	2	0.64516129	0.279	Grey shale rock	Greyish	8.39	6.29	4	1100	8	25.0298
9:00	11:00	2:00	120	43	44.5	1.5	1.25	0.304	Grey shale rock	Greyish	16.67	0	0	1100	6	100
11:30	13:30	2:00	120	44.5	46	1.5	1.25	0.457	Grey shale rock	Greyish	16.67	0	0	1100	6	100
14:00	16:00	2:00	120	46	47.5	1.5	1.25	0.736	Grey shale rock	Greyish	18.33	0	0	1100	6	100
16:00	13:00	3:00	180	47.5	49	1.5	0.83333333	0.762	Grey shale rock	Greyish	16.67	0	0	1000	6	100
13:30	15:00	1:30	90	49	50	1	1.11111111	0.406	Grey shale rock	Greyish	22.22	0	0	1000	6	100
							0.7732051	0.20138								86.4435

Interpretation and basis for design

Based on GVAC shown in Appendix-3, after the drilling of GL-34.0 m and deeper, the goundwater levels before drilling were observed as stable. In addition, leackage of drilling water was improved after the drilling of GL-34.0 m and deeper as well. Thus at the depth of GL-34.0 m and deeper, it is highly likely that less loosened bedrock lies. The depths of slip surface was, threfore, set at GL-34.0 m.

The highest groundwater level during the groundwater monitoring was observed at GL-0.47 m on the 7th of May, 2021. Thus GL-0.47 m was set as HWL.

Appendix 3

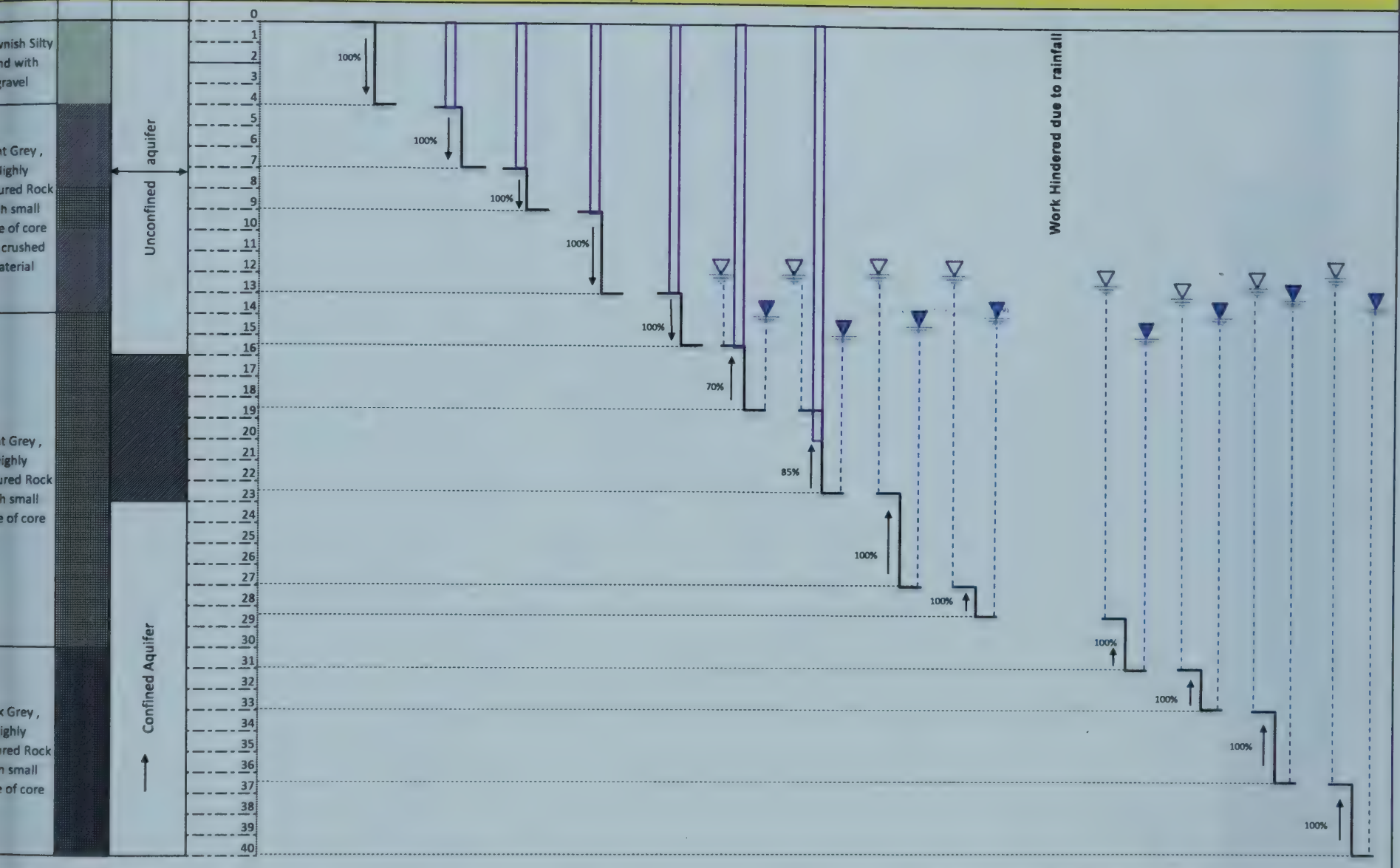
Groundwater Variation Analysis Chart (GVAC)

Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		**Project Name**	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE
Date Of ommencement	22 March 2021	**Location**	163+550
e Of Completion	04 April 2021	**Ground Elevation(m)**	597.00
BoreHole No.	BV-1	**Job No.**	thoughts/2020-21/BV01
BoreHole Depth	40m		

hology	**LIMO**	**Class**	**Depth(m)**	**Date**	**22.2.21**	**23.3.21**	**24.3.12**	**25.3.21**	**26.3.21**	**27.3.21**	**28.3.21**	**29.3.21**	**30.3.21**	**31.3.21**	**01.4.21**	**02.4.21**	**03.4.21**	**04.4.21**
				Weather(W)	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Cloudy	Rainy	Clear	Clear	Clear	Clear
					Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Rainy	Rainy	Sunny	Sunny	Sunny	Sunny

W: Weather (of Last night / of the day)

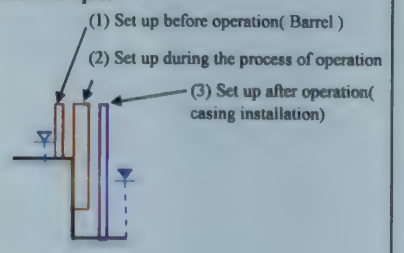


Classification	
⟷	**Permeable layer**
	Partial Leakage
	Least Leakage
↑	**Pressure water**
	Aquiclude
?	Unknown

Legend

	Excavation Section		
	Water level before operation		Casing pipe
	Water level after operation		
d	Dry Excavation section		
↑	Water gushing zone		
↓	Leakage Zone		

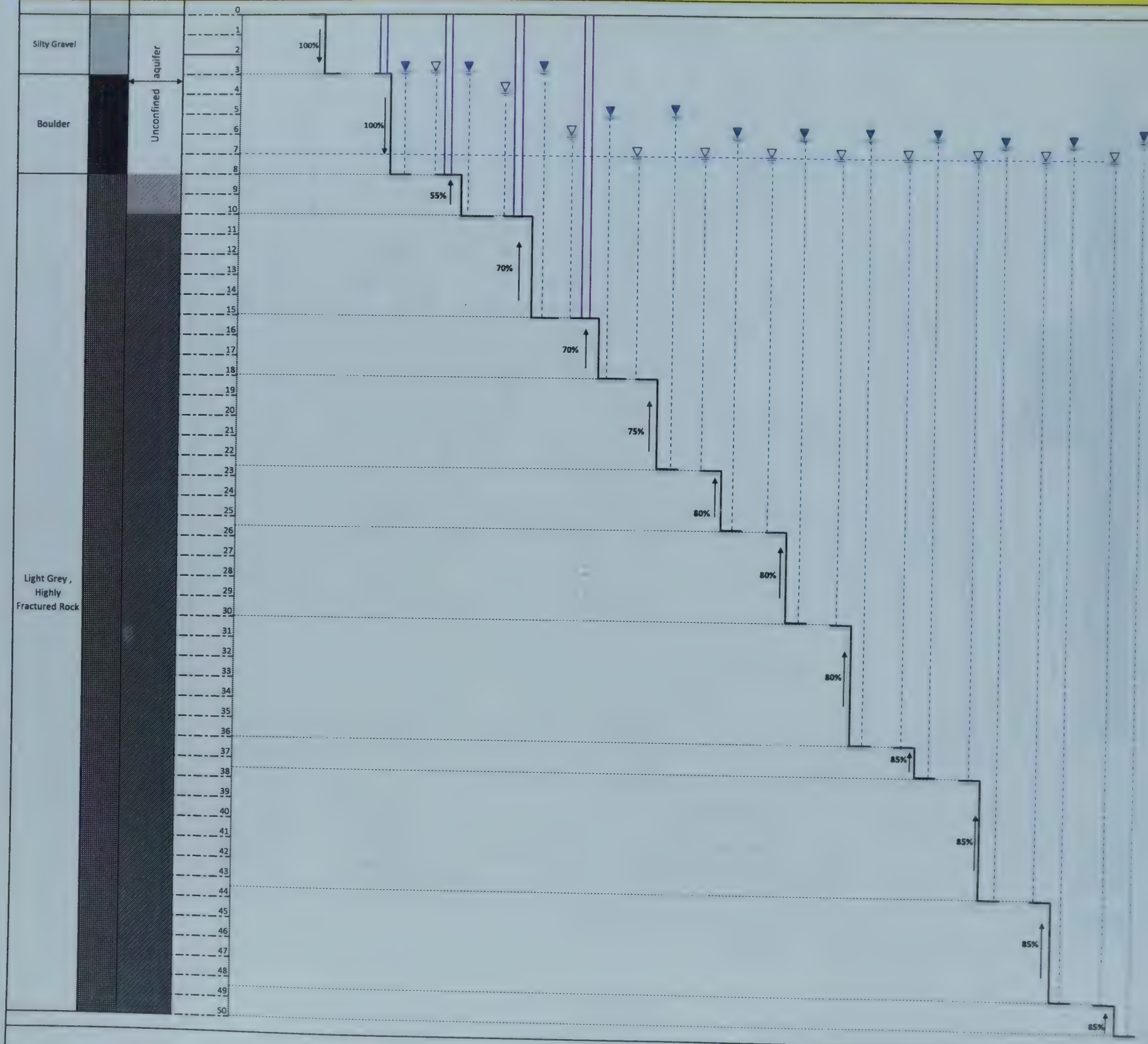
Case Example:



Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		**Project Name**	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Date Of Commencement	29 August 2021		**Location**	Latitude	Longitude	25.12831166 93.03647500				**BoreHole No.**	BV-2	**Job No.**	thoughts/2020-21/BV02			
Date Of Completion	10 September 2021		**Ground Elevation(m)**	640.00						**BoreHole Depth**	50 m					
Lithology	**LIMO**	**Class**	**Depth(m)**	**Date**	29.8.21	30.8.21	31.8.21	01.9.21	02.9.21	03.9.21	04.9.21	05.9.21	06.9.21	07.9.21	08.9.21	09.9.21	10.09.2
				Weather(W)	Clear	Clear	Rainy	Clear	Rainy	Clear	Clear	Clear	Clear	Rainy	Clear	Clear	Clear
					Clear	Rainy	Rainy	Rainy	Clear	Clear	Clear	Clear	Clear	Clear	Rainy	Clear	Clear
				W: Weather →					of Last night of the day								

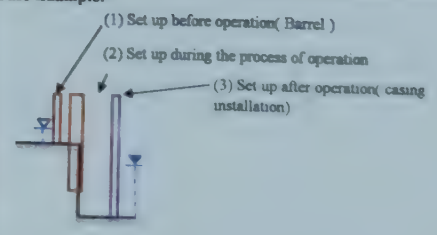


Classification	
↔	**Permeable layer**
	Partial Leakage
	Least Leakage
↑	**Pressure water**
	Aquiclude
?	Unknown

Legend

	Excavation Section		
	Water level before operation		Casing pipe
	Water level after operation		
d	Dry Excavation section		
↑	Water gushing zone		
↓	Leakage Zone		

Case Example:



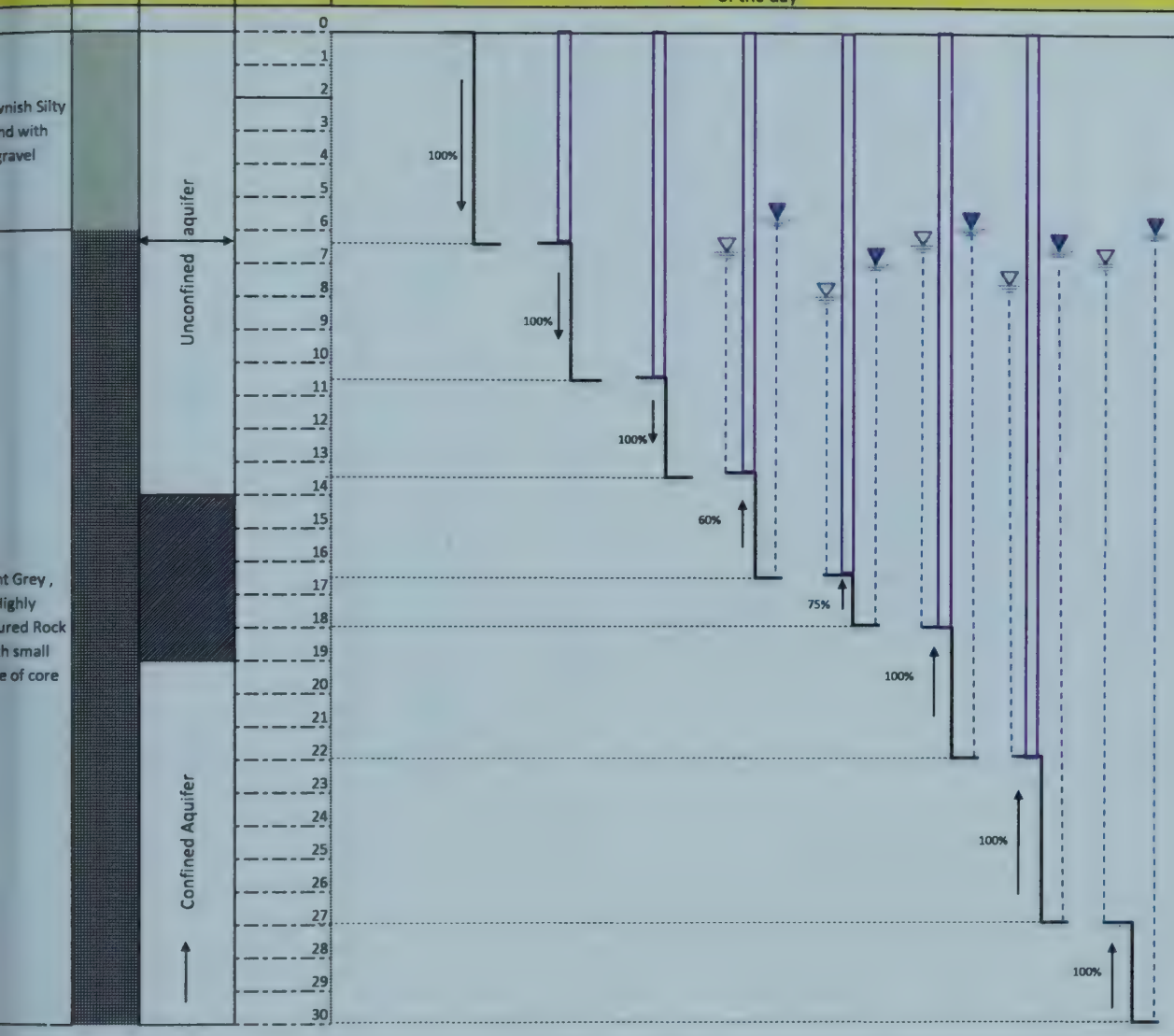
Groundwater Variation Analysis Chart

	THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR	Project Name	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Date Of ommencement	12 April 2021	Location	163+575	BoreHole No.	BV-3	Job No.	thoughts/2020-21/BV03
e Of Completion	19 April 2021	Ground Elevation(m)	560.00	BoreHole Depth	30m		

hology	LIMO	Class	Depth(m)	Date	12.4.21	13.4.21	14.4.21	15.4-21	16.4.21	17.4.21	18.4.21	19.4.21
				Weather(W)	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear
					Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny

W: Weather — of Last night / of the day



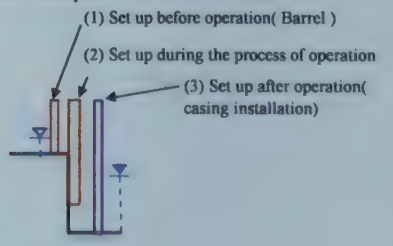
Classification	
←→	Permeable layer
	Partial Leakage
	Least Leakage
↑	Pressure water
	Aquiclude
?	Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

Case Example:

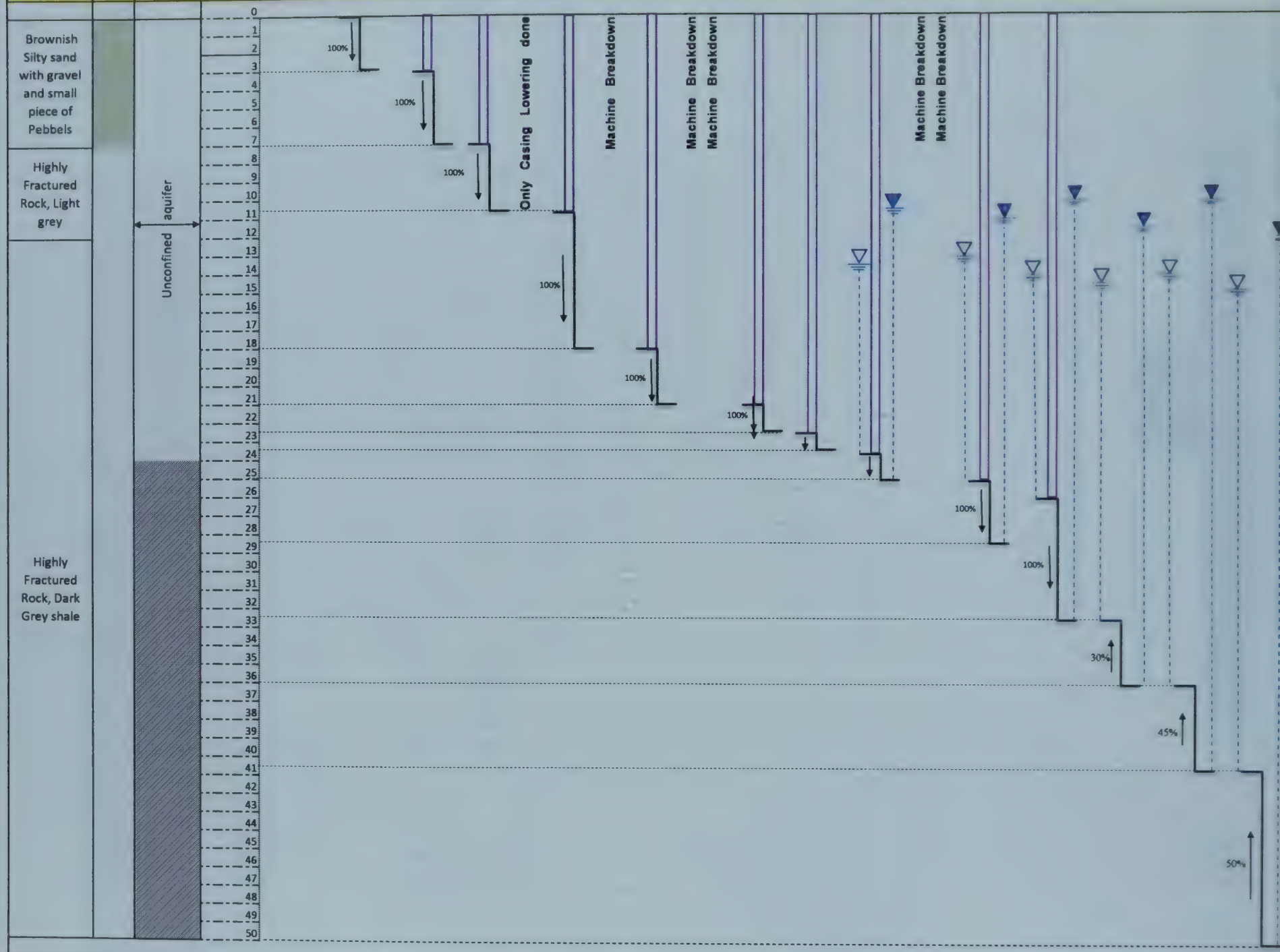


Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR	Project Name	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE	

Date Of Commencement	31 January 2021	Location	163+650			BoreHole No.	BV-4		Job No.	thoughts/2020-21/BV04										
Date Of Completion	18 February 2021	Ground Elevation(m)	591.00			BoreHole Depth	50m													
Lithology	LIMO	Class	Depth(m)	Date	31.1.21	01.1.21	02.1.21	03.2.21	04.2.21	05.2.21	06.2.21	07.2.21	08.2.21	09.2.21	10.2.21	11.2.21	12.2.21	13.2.21	14.2.21	15.2.21	16.2.21	17.2.21	18.2.21
				Weather(W)	Clear	Clear	Clear		Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear
					Sunny	Sunny	Sunny		Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny

W: Weather — of Last night / of the day



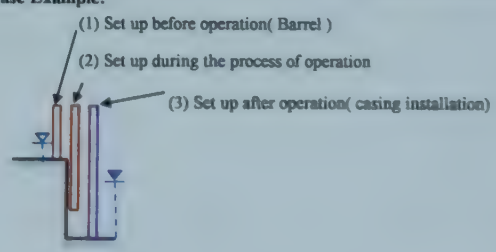
Classification
Permeable layer
Partial Leakage
Least Leakage
Pressure water
Aquiclude
? Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
	Water gushing zone
	Leakage Zone

Casing

Case Example:



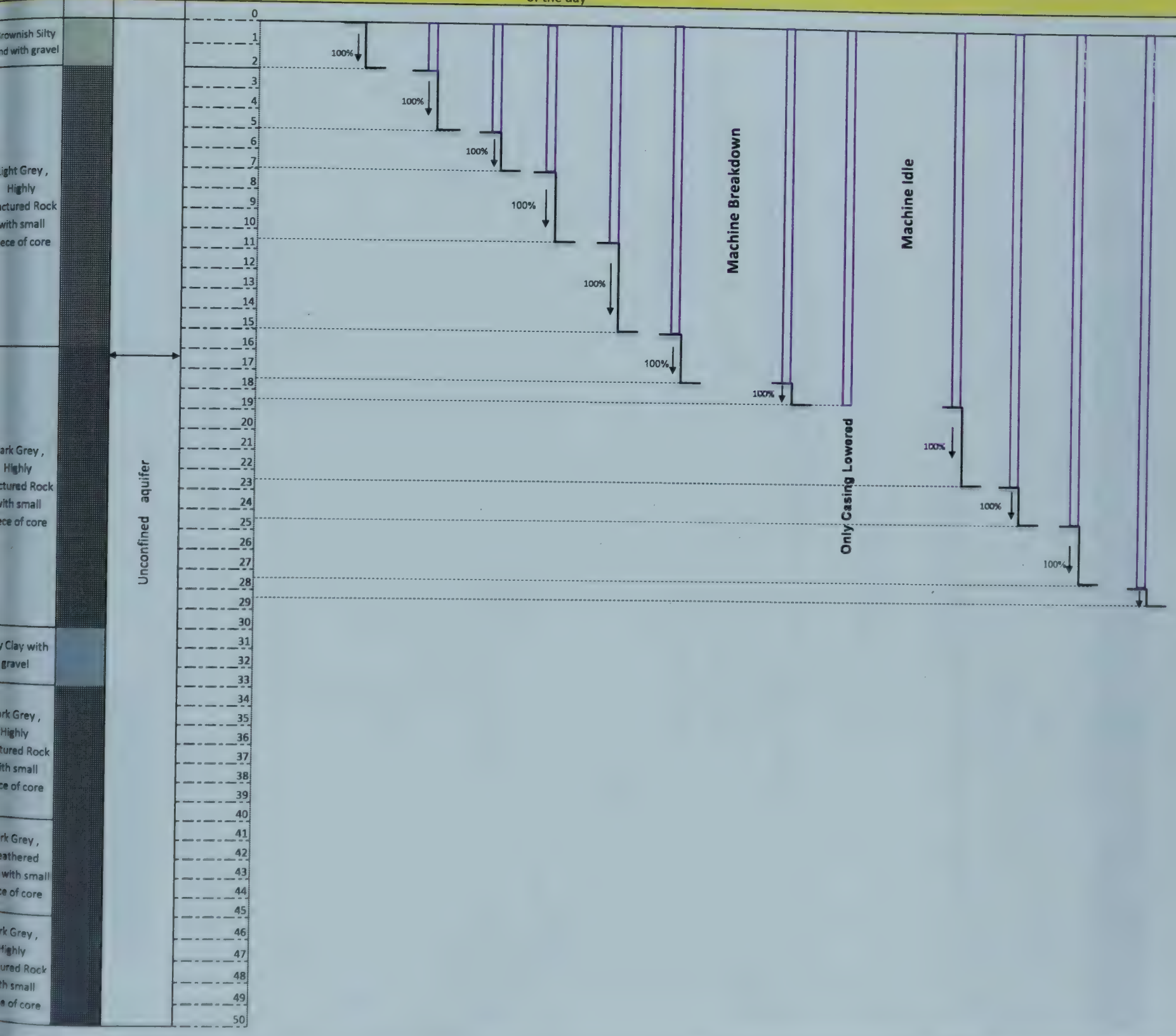
Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR	**Project Name**	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE	

Date Of Commencement	06 May 2021	Location	163+00	BoreHole No.	BV-5	Job No.	thoughts/2020-21/BV05										
Date Of Completion	25 May 2021	Ground Elevation(m)	639.00	BoreHole Depth	50 m												

Lithology	LIMO	Class	Depth(m)	Date	05.5.21	06.5.21	07.5.21	08.5.21	09.5.21	10.5.21	11.5.21	12.5.21	13.5.21	14.5.21	15.5.21	16.5.21	17.5.21	18.5.21
				Weather(W)	Cloudy	Rainy	Rainy	Cloudy	Rainy	Rainy	Clear	Clear	Clear	Clear	Cloudy	Rainy	Clear	Rainy
					Rainy	Cloudy	Clear	Clear	Clear	Rainy	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Rainy

W: Weather — of Last night / of the day



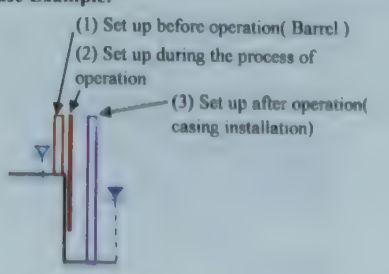
Classification	
↔	**Permeable layer**
	Partial Leakage
	Least Leakage
↑	**Pressure water**
	Aquiclude
?	Unknown

Legend

	Excavation Section
▽	Water level before operation
▼	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

Case Example:



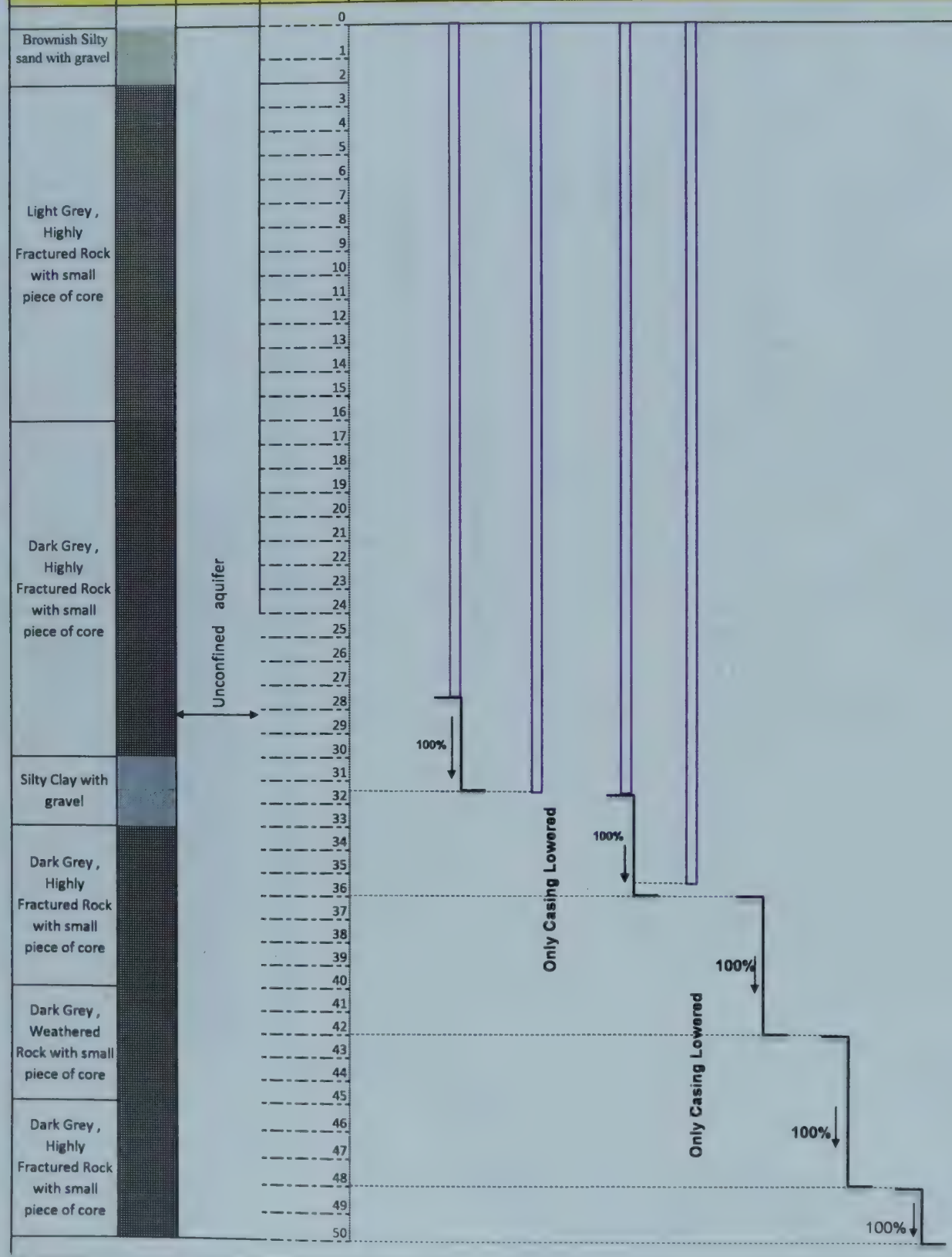
Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		Project Name	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANG TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Date Of Commencement	06 May 2021	Location	163+00	BoreHole No.	BV-5	Job No.	thoughts/2020-21/BV05
Date Of Completion	25 May 2021	Ground Elevation(m)	639.00	BoreHole Depth	50 m		

Lithology	LIMO	Class	Depth(m)	Date	19.5.21	20.5.21	21.5.21	22.5.21	23.5.21	24.5.21	25.5.21
				Weather(W)	Cloudy	Cloudy	Cloudy	Cloudy	Clear	Cloudy	Cloudy
					Clear	Clear	Clear	Clear	Clear	Clear	Clear

W: Weather — of Last night / of the day



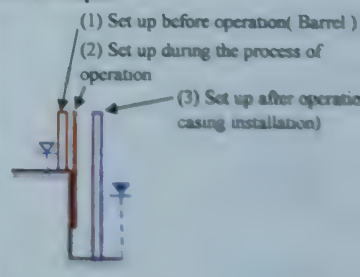
Classification	
↔	Permeable layer
	Partial Leakage
	Least Leakage
↑	Pressure water
	Aquiclude
?	Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

Case Example:

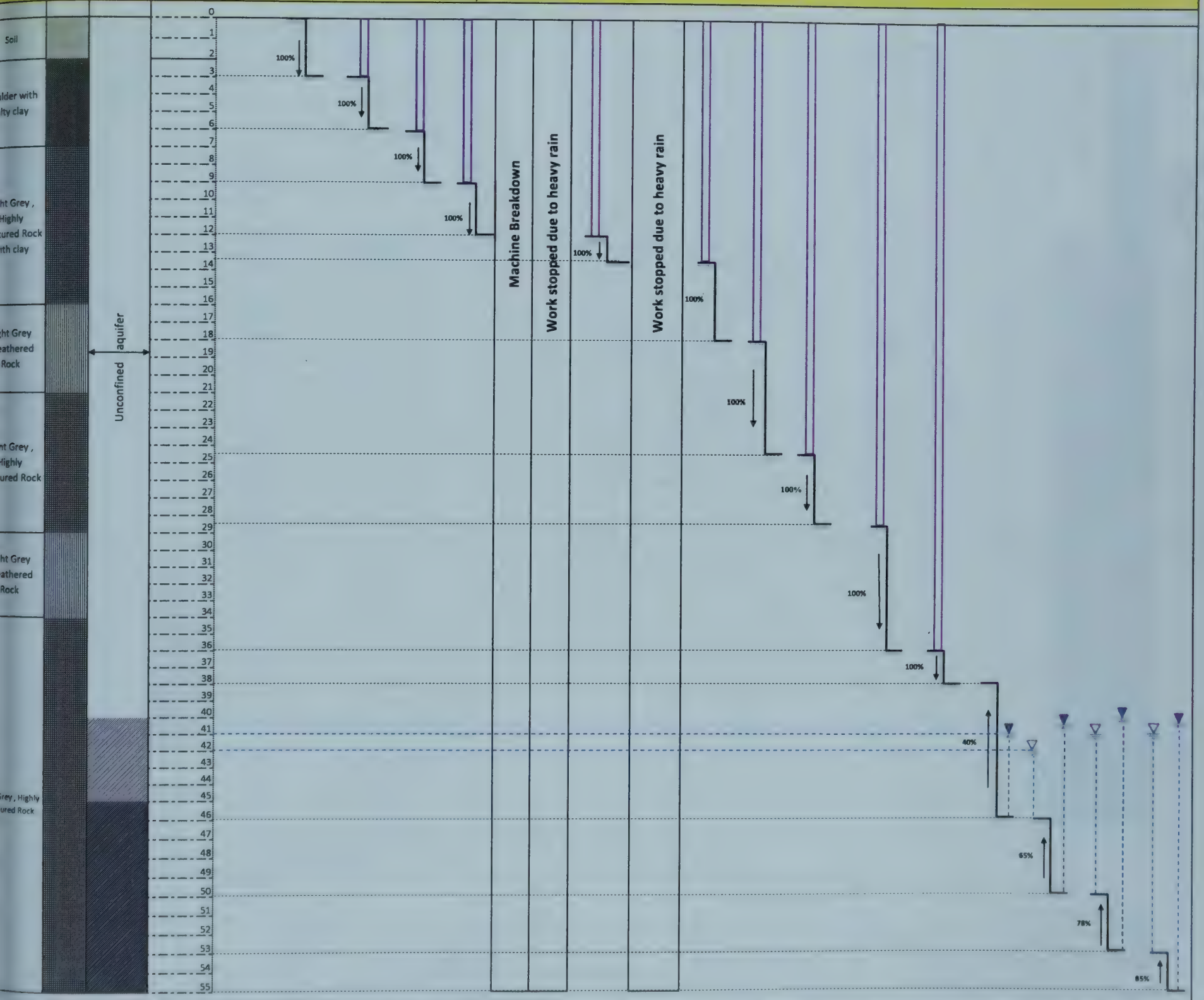


Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		**Project Name**	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE
Date Of Commencement	23 June 2021	**Location**	
Date Of Completion	On progress	**Ground Elevation(m)**	702.00
BoreHole No.	BV-6	**Job No.**	thoughts/2020-21/BV06
BoreHole Depth	55 m		

Lithology	LIMO	Class	Depth(m)	Date	23.6.21	24.6.21	25.6.21	26.6.21	27.6.21	28.6.21	29.6.21	30.6.21	01.7.21	02.7.21	03.7.21	04.7.21	05.7.21	06.7.21	07.07.21	08.07.21	09.07.21	10.07.21	13.07.21
				Weather(W)	Clear	Clear	Clear	Clear	Clear	Clear	rainy	rainy	Clear	rainy	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear
					Clear	Clear	Clear	Clear	Clear	Clear	rainy	rainy	rainy	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear

W: Weather — of Last night / of the day



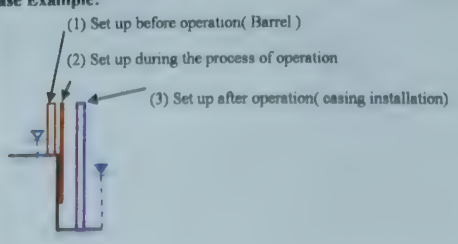
Classification	
←→	Permeable layer
	Partial Leakage
	Least Leakage
↑	Pressure water
	Aquiclude
?	Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

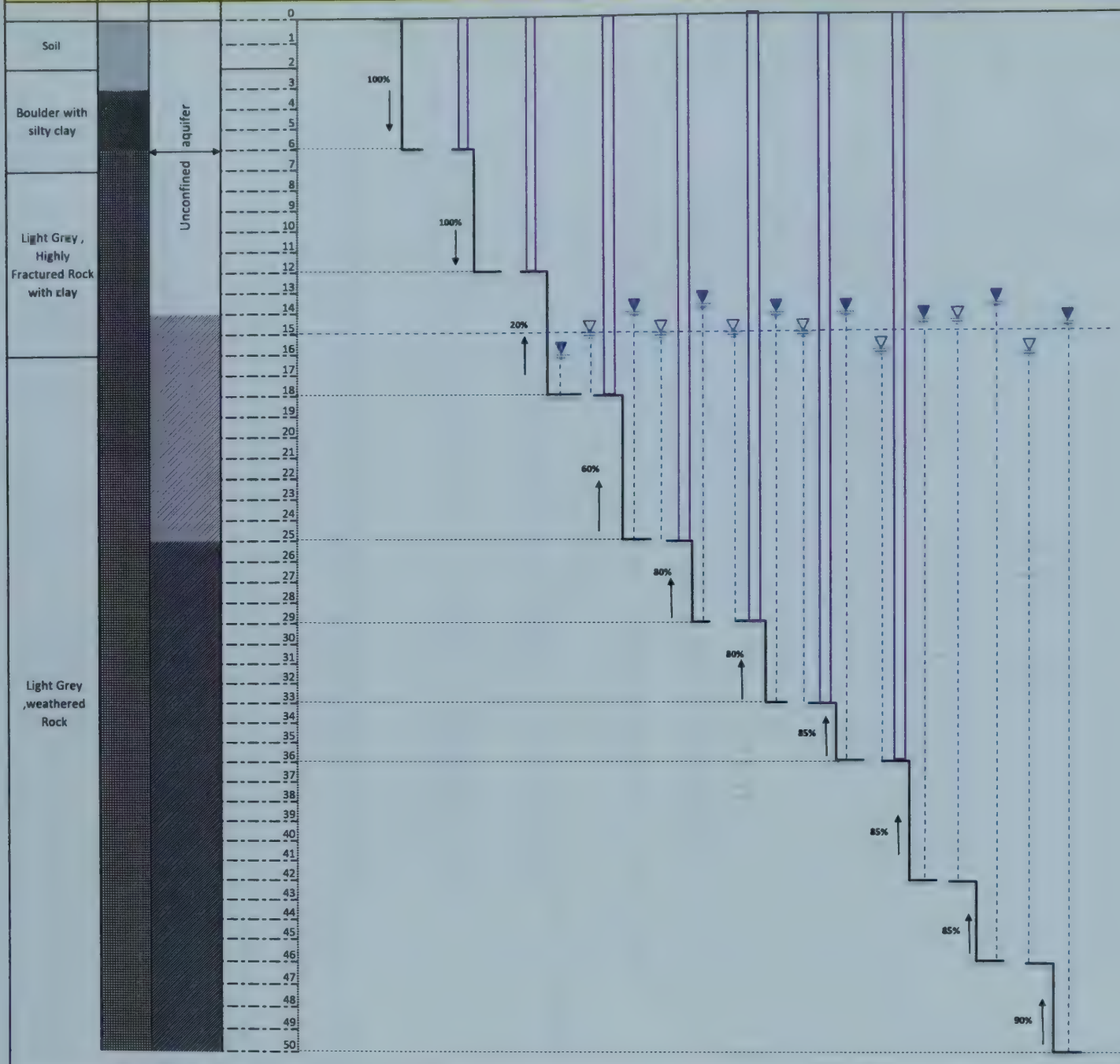
Case Example:



Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		**Project Name**	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE

Date Of Commencement	18 July 2021		**Location**	E- 503655.893 N- 2779056.8652		**BoreHole No.**					BV-7			**Job No.**	thoughts/2020-21/BV07			
Date Of Completion	28 July 2021		**Ground Elevation(m)**	707.00		**BoreHole Depth**					50 m							
Lithology	**LIMO**	**Class**	**Depth(m)**	**Date**	18.7.21	19.7.21	20.7.21	22.7.21	23.7.21	24.7.21	25.7.21	26.7.21	27.7.21	28.7.21						
				Weather(W)	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear						
					Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear	Clear						
				W: Weather →			of Last night / of the day													



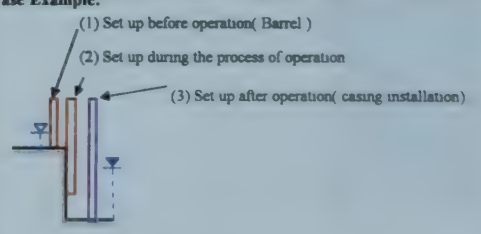
Classification	
↔	**Permeable layer**
	Partial Leakage
	Least Leakage
↑	**Pressure water**
	Aquiclude
?	Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

Case Example:

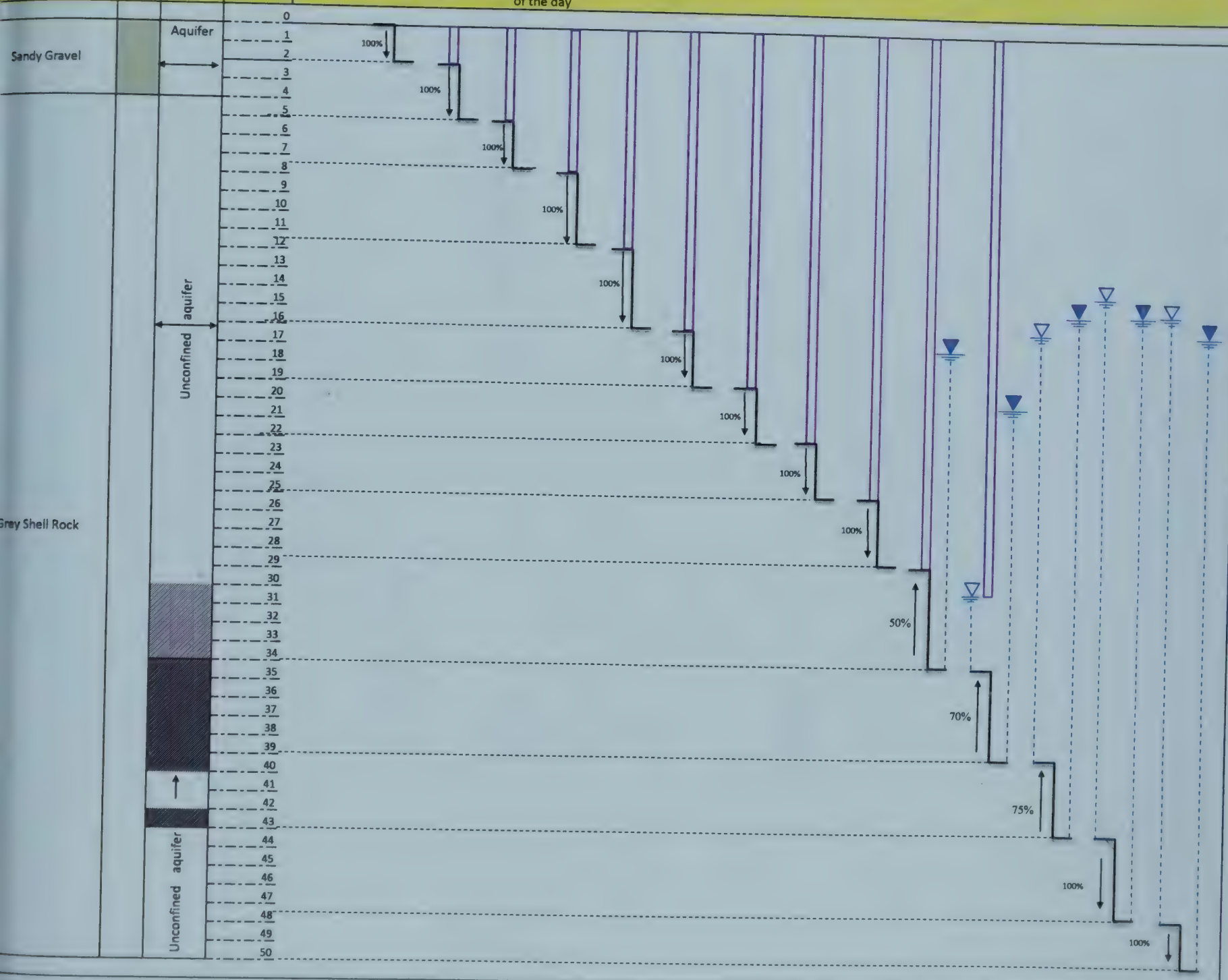


Groundwater Variation Analysis Chart

THOUGHTS CONSULTANTS JAIPUR PVT. LTD., JAIPUR		Project Name	CONSULTANCY SERVICE FOR PREPARATION OF DPR FOR BALANCE WORK OF CONSTRUCTION OF 4 LANING ROAD FROM KM 140.700 to 165.080 (NARIMBANGLO TO JATINGA SECTION), KM 165.220 TO 190.587 (JATINGA JUNCTION TO HARANGAJO) OF NH 54 IN THE STATE OF ASSAM UNDER PHASE II PROGRAM OF NHDP ON EPC MODE
Date Of Commencement	11 January 2021	Location	CH- 163.825
Date Of Completion	26 January 2021	Ground Elevation	604.00
BoreHole No.	BV-10	Job No.	thoughts/2020-21/BV10
BoreHole Depth	50m		

Lithology	LIMO	Class	Depth(m)	Date	11.1.21	12.1.21	13.1.21	14.1.21	15.1.21	16.1.21	17.1.21	18.1.21	19.1.21	20.1.21	21.1.21	22.1.21	25.1.21	26.1.21
				Weather(W)	Clear	Clear	Clear	Clear	Cloudy	Clear	Cloudy	Cloudy	Cloudy	Clear	Clear	Clear	Clear	Clear
					Sunny	Sunny	Sunny	Sunny	Sunny	Sunny	Rainy	Rainy	Sunny	Sunny	Sunny	Sunny	Sunny	Sunny

W: Weather — of Last night / of the day



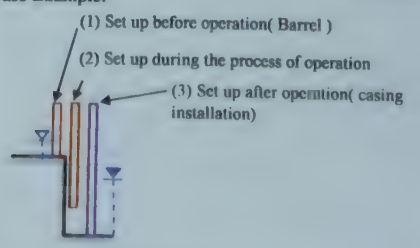
Classification	
↔	Permeable layer
	Partial Leakage
	Least Leakage
↑	Pressure water
	Aquiclude
?	Unknown

Legend

	Excavation Section
	Water level before operation
	Water level after operation
d	Dry Excavation section
↑	Water gushing zone
↓	Leakage Zone

Casing pipe

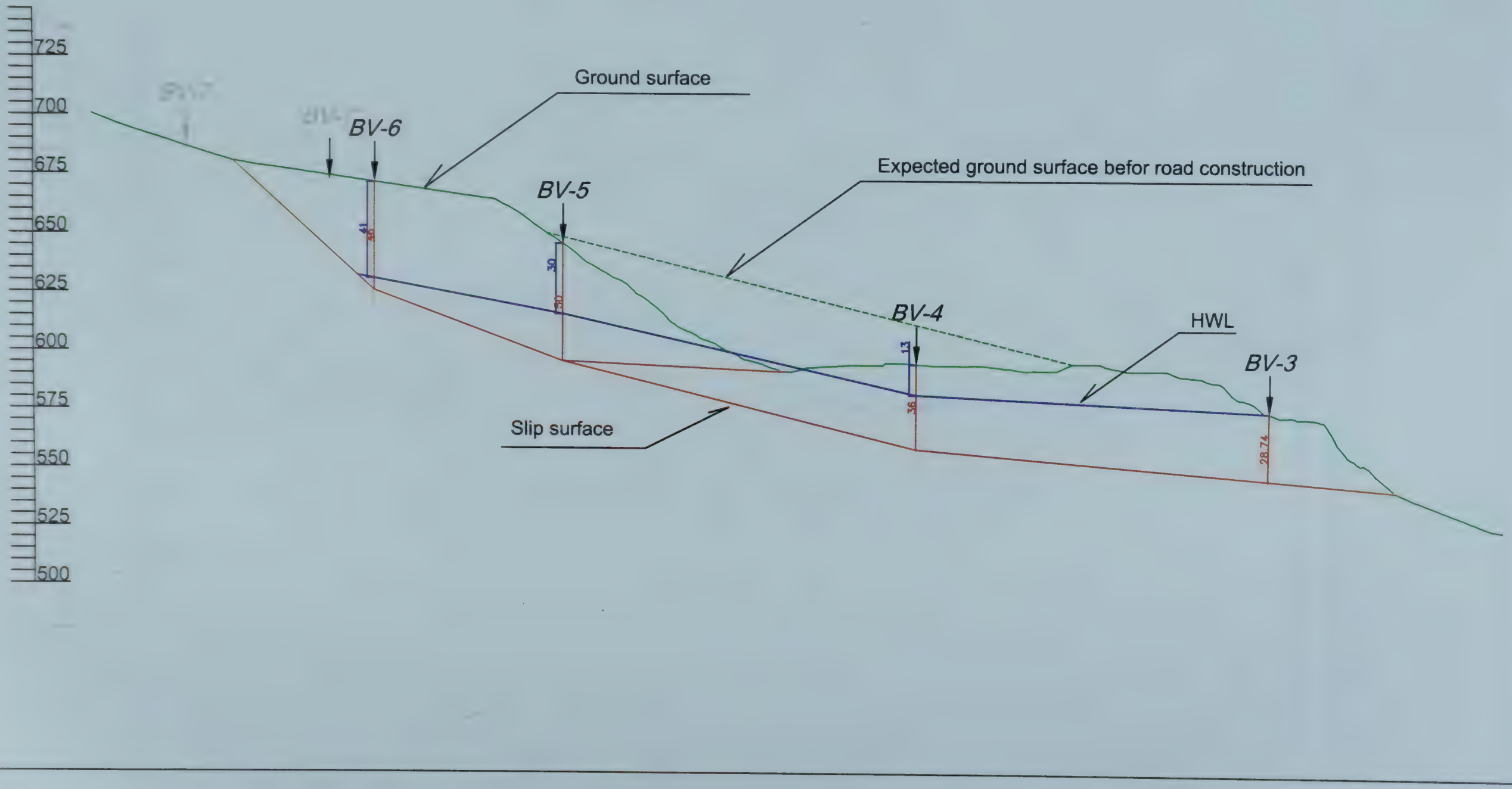
Case Example:



Appendix 4

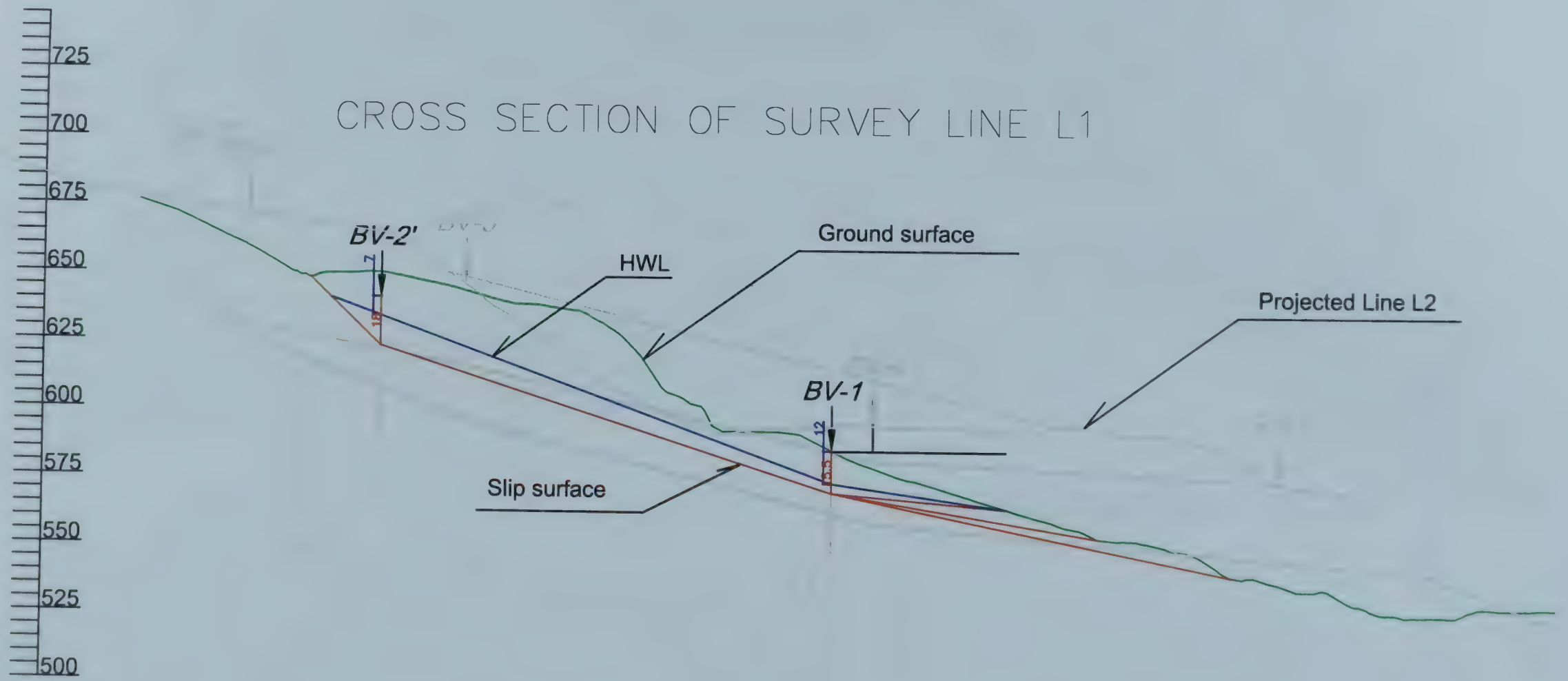
Geological Profile for Stability Analysis

CROSS SECTION OF SURVEY LINE L2



A03-51

CROSS SECTION OF SURVEY LINE L1



A03-52

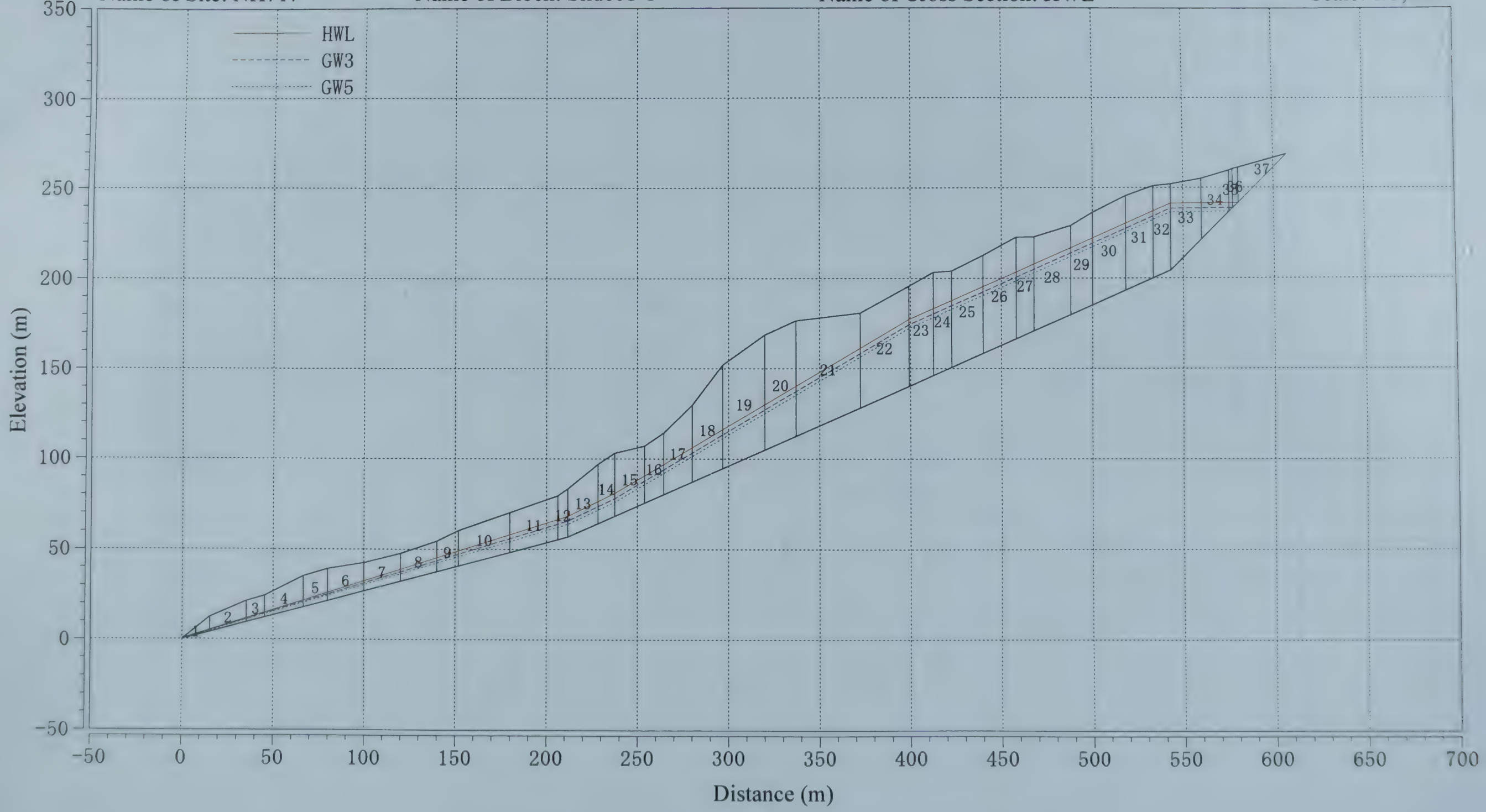
Appendix 5-1

Results of Stability Analysis

Figures and Tables with English Translation

Cross Section

Name of Site: NH717 Name of Block: Slide01-1 Name of Cross Section: HWL Scale: 1/3,000



A03-54

Correlation Diagram of C'- ϕ'

Name of Site: NH717

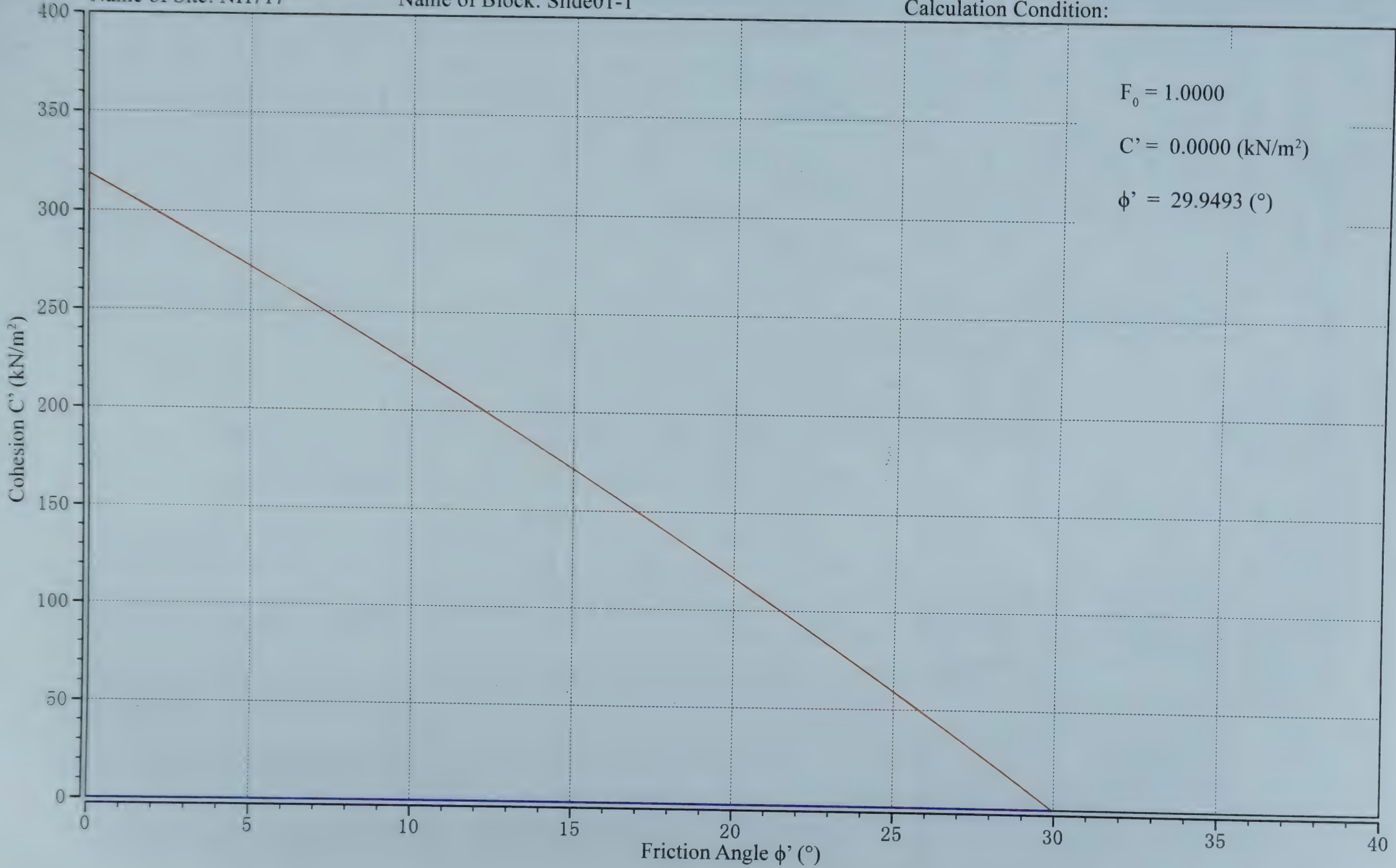
Name of Block: Slide01-1

Calculation Condition:

F_0 = 1.0000

C' = 0.0000 (kN/m²)

ϕ' = 29.9493 (°)



A03-55

Slidel whole dr ainage & remova

Spreadsheet for Two-dimensional Stability Analysis

Site	NH717	Block	Slide01-1	Analysis Method	Simplified Janbu Method	Scarplet at Downslope Side of Depression Zone	No
Cross Section	HWL	Pore Wter Pressure	HWL	Groundwater Condition	Confined		
Calculation Condition				Horizontal Component of External Force Q (kN/m)	0.00	Horizontal Component of Seismic Intensity k_h	
				Reservoir Water	None	Elevation of Reference Water Surface BL (m)	
				Coordinate of Reference Point	x= , y= , z=		

Geotechnical Parameters		Mean Value
Wet Unit Weight of Sliding Mass	γ (kN/m^3)	25.000
Submerged Unit Weight of Sliding Mass	γ' (kN/m^3)	
Saturated Unit Weight of Sliding Mass	γ_{sat} (kN/m^3)	
Friction Angle of Depression Zone	ϕ_e (°)	
Cohesion of Slip Surface	c' (kN/m^2)	0.0000
Friction Angle of Slip Surface	ϕ' (°)	29.9493
Maximum Depth of Arc or Slip Surface	d (m)	33.78
Length of Chord of Arc or Slip Surface	L (m)	664.42
Correction Factor	f_0	1.0282
Inclination of Main Scarp	θ_u (°)	
Inclination of Scarplet at Downslope Side of	θ_d (°)	
Area of Sliding Mass	A (m^2)	21669.97
Shear Force of Landslide	Σ S (kN/m)	239971.29
Driving Force of Landslide	Σ T (kN/m)	246726.34

Designed Bearing Force				
Numerator	Groundanchors	P_{sa}	(kN/m)	
	Piles	P_{sp}	(kN/m)	
	Others	P_{sx}	(kN/m)	
Denominator	Groundanchors	P_{ma}	(kN/m)	
	Piles	P_{mp}	(kN/m)	
	Others	P_{mx}	(kN/m)	
Piezometric Height of Groundwater of Depression Zone		Z_w	(m)	
Horizontal componet of water presssure		Q_w	(kN/m)	
Depth of Slip Surface at Downslope Side of Depression Zone		Z_h	(m)	
Coefficient of Active Earth Pressure		k_a		
Active Earth Pressure		E_a	(kN/m)	
Two-dimensional Factors of Safety		F_2		1.0000

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

Slice	Depth of Slip Surface at Downslope Side h_1 (m)	Depth of Slip Surface at Upslope Side h_2 (m)	Horizontal Width of Slice b (m)	Inclination of Slip Surface α (°)	Weight W (kN/m)	Slice	Depth of Slip Surface at Downslope Side h_1 (m)	Depth of Slip Surface at Upslope Side h_2 (m)	Horizontal Width of Slice b (m)	Inclination of Slip Surface α (°)	Weight W (kN/m)
1	0.000	8.270	15.10	15.000	1560.96	36	21.900	19.620	3.02	45.000	1567.38
2	8.270	11.740	20.13	15.000	5035.02	37	19.620	0.000	27.24	45.000	6680.61
3	11.740	12.020	10.06	15.000	2987.82						
4	12.020	17.250	21.38	15.000	7822.41						
5	17.250	17.680	13.33	15.000	5820.21						
6	17.680	15.700	20.00	15.000	8345.00						
7	15.700	15.350	20.00	15.000	7762.50						
8	15.350	16.950	20.00	15.000	8075.00						
9	16.950	19.590	11.77	15.000	5375.95						
10	19.590	21.900	28.23	15.000	14640.78						
11	21.900	24.090	26.31	15.000	15124.96						
12	24.090	26.510	5.49	15.000	3472.43						
13	26.510	32.830	16.62	24.000	12327.89						
14	32.830	35.000	9.11	24.000	7724.14						
15	35.000	31.650	16.36	24.000	13629.93						
16	31.650	34.150	10.50	24.000	8636.25						
17	34.150	42.630	15.62	24.000	14991.30						
18	42.630	57.400	16.88	24.000	21106.33						
19	57.400	63.810	23.12	24.000	35029.69						
20	63.810	64.070	17.13	24.000	27382.31						
21	64.070	52.520	35.22	24.000	51328.75						
22	52.520	55.500	26.78	24.000	36159.70						
23	55.500	57.060	13.48	24.000	18966.36						
24	57.060	53.390	10.06	24.000	13889.09						
25	53.390	54.350	17.32	24.000	23325.71						
26	54.350	56.170	18.12	24.000	25032.78						
27	56.170	51.910	9.84	24.000	13293.84						
28	51.910	49.100	20.13	24.000	25416.64						
29	49.100	51.120	11.91	24.000	14920.25						
30	51.120	52.240	18.28	24.000	23617.76						
31	52.240	51.000	15.10	24.000	19486.55						
32	51.000	47.580	9.75	24.000	12014.44						
33	47.580	33.820	16.87	45.000	17165.23						
34	33.820	22.980	15.36	45.000	10905.60						
35	22.980	21.900	2.01	45.000	1127.61						

Note:

Slide1_whole_dr ainage & remova

Spreadsheet for Two-dimensional Stability Analysis

Site	NH717	Block	Slide01-1	Analysis Method	Simplified Janbu Method	Scarplet at Downslope Side	No.
Cross Section	HWL	Pore Wter Pressure	HWL	Groundwater Condition	Confined		
Calculation Condition				Horizontal Component of Reservoir Water	0.00	Horizontal Component of Elevation of Reference Water Surface	
					None		
Geotechnical Parameters		Mean Value		Coordinate of Reference Point (m)	x= , y= , z=		
Wet Unit Weight of Sliding Mass	γ (kN/m³)	25.000		Designed Bearing Force	Numerator	Groundanchors P_{sa} (kN/m)	
Submerged Unit Weight of Sliding Mass	γ' (kN/m³)					Piles P_{sp} (kN/m)	
Saturated Unit Weight of Sliding Mass	γ_{sat} (kN/m³)					Others P_{sx} (kN/m)	
Friction Angle of Depression Zone	ϕ_s (°)				Denominator	Groundanchors P_{na} (kN/m)	
Cohesion of Slip Surface	c' (kN/m²)	0.0000				Piles P_{np} (kN/m)	
Friction Angle of Slip Surface	ϕ' (°)	29.9493				Others P_{nx} (kN/m)	
Shear Force of Landslide	Σ S (kN/m)	239971.29		Area of Sliding Mass	A (m²)		21669.97
Driving Force of Landslide	Σ T (kN/m)	246726.34		Two-dimensional Factors of Safety	F_2		1.0000

Slice	Piezometric Height above Bottom of Slice U_2 (m)	Horizontal Width of Slice b_n (m)	Component Force in Normal Direction of Slip Surface N (kN/m)	Inclination of Seepage Pressure θ_p (°)	Seepage Pressure P (kN/m)	Pore Wter Pressure U (kN/m)	Length of Slip Surface L (m)	Inclination of Interslice Force from Horizontal α_i (°)	Vertical Shear Force Δx (kN/m)	Shear Force of Landslide s_1 (kN/m)	Shear Force of Landslide n_α	Shear Force of Landslide $S = s_1 / n_\alpha$ (kN/m)	Driving Force of Landslide T (kN/m)
1	0.810	15.10				60.0	15.63			864.83	1.0771	802.95	418.26
2	1.880	20.13				265.5	20.84			2748.05	1.0771	2551.45	1349.13
3	2.420	10.06				212.1	10.41			1599.28	1.0771	1484.87	800.58
4	3.560	21.38				626.9	22.13			4145.84	1.0771	3849.24	2096.01
5	4.280	13.33				512.4	13.80			3058.19	1.0771	2839.40	1559.52
6	5.350	20.00				944.4	20.71			4264.02	1.0771	3958.96	2236.04
7	6.420	20.00				1154.2	20.71			3807.49	1.0771	3535.09	2079.96
8	7.490	20.00				1364.1	20.71			3866.62	1.0771	3589.99	2163.69
9	8.110	11.77				900.3	12.19			2578.73	1.0771	2394.24	1440.48
10	9.620	28.23				2454.2	29.23			7021.56	1.0771	6519.21	3922.99
11	11.030	26.31				2664.0	27.24			7179.65	1.0771	6666.00	4052.72
12	11.320	5.49				601.6	5.68			1654.06	1.0771	1535.72	930.43
13	12.210	16.62				1917.5	18.19			5998.14	1.0487	5719.84	5488.73
14	12.700	9.11				1112.7	9.97			3809.31	1.0487	3632.57	3439.01
15	15.160	16.36				2234.9	17.91			6565.49	1.0487	6260.87	6068.43
16	16.740	10.50				1642.4	11.49			4029.67	1.0487	3842.71	3845.11
17	19.090	15.62				2744.2	17.10			7056.41	1.0487	6729.01	6674.55
18	21.620	16.88				3369.5	18.48			10219.45	1.0487	9745.30	9397.14
19	25.100	23.12				5296.4	25.31			17131.46	1.0487	16336.61	15596.22
20	27.680	17.13				4433.2	18.75			13222.61	1.0487	12609.12	12191.39
21	32.970	35.22				10474.0	38.55			23539.35	1.0487	22447.19	22853.03
22	37.000	26.78				9187.8	29.31			15540.41	1.0487	14819.37	16099.33
23	37.000	13.48				4891.2	14.76			8109.72	1.0487	7733.45	8444.37
24	37.000	10.06				3650.2	11.01			5899.33	1.0487	5625.62	6183.82
25	37.000	17.32				6284.5	18.96			9818.66	1.0487	9363.10	10385.28
26	37.000	18.12				6574.8	19.83			10634.97	1.0487	10141.54	11145.31
27	37.000	9.84				3570.4	10.77			5602.36	1.0487	5342.43	5918.80
28	37.000	20.13				7304.1	22.04			10435.93	1.0487	9951.73	11316.22
29	37.000	11.91				4321.5	13.04			6106.70	1.0487	5823.36	6642.92
30	37.000	18.28				6632.8	20.01			9786.23	1.0487	9332.17	10515.30
31	37.000	15.10				5479.0	16.53			8070.76	1.0487	7696.30	8675.97
32	37.000	9.75				3537.7	10.67			4884.02	1.0487	4657.42	5349.17
33	20.250	16.87				4735.7	23.86			7161.55	0.7881	9087.28	17165.23
34	5.000	15.36				1901.7	21.72			5187.78	0.7881	6582.77	10905.60
35	3.000	2.01				78.8	2.84			604.27	0.7881	766.75	1127.61
36	0.000	3.02				44.4	4.27			877.48	0.7881	1113.44	1567.38
37						0.0	38.52			3849.17	0.7881	4884.21	6680.61

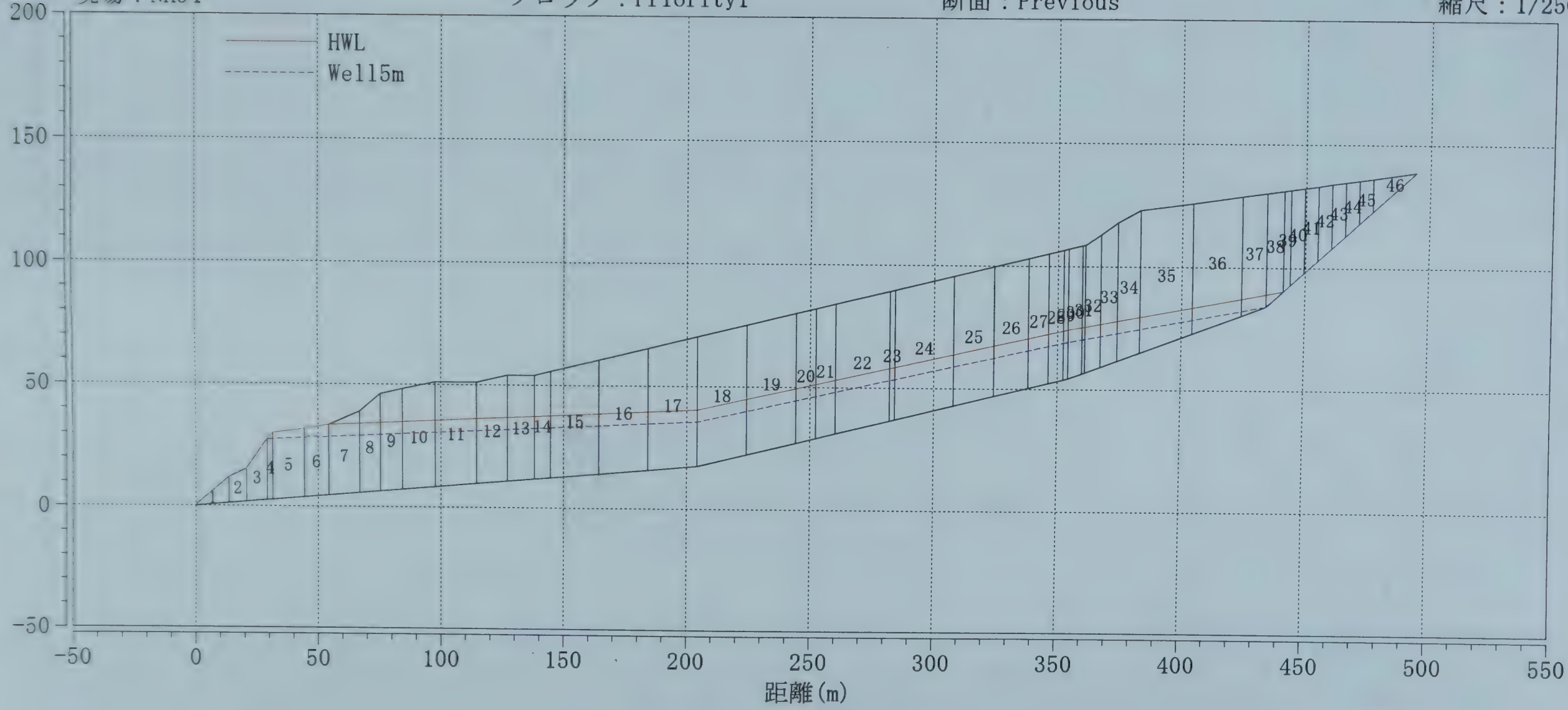
Appendix 5-2

Results of Stability Analysis

Main Survey Line L2

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Previous　　縮尺：1/2500

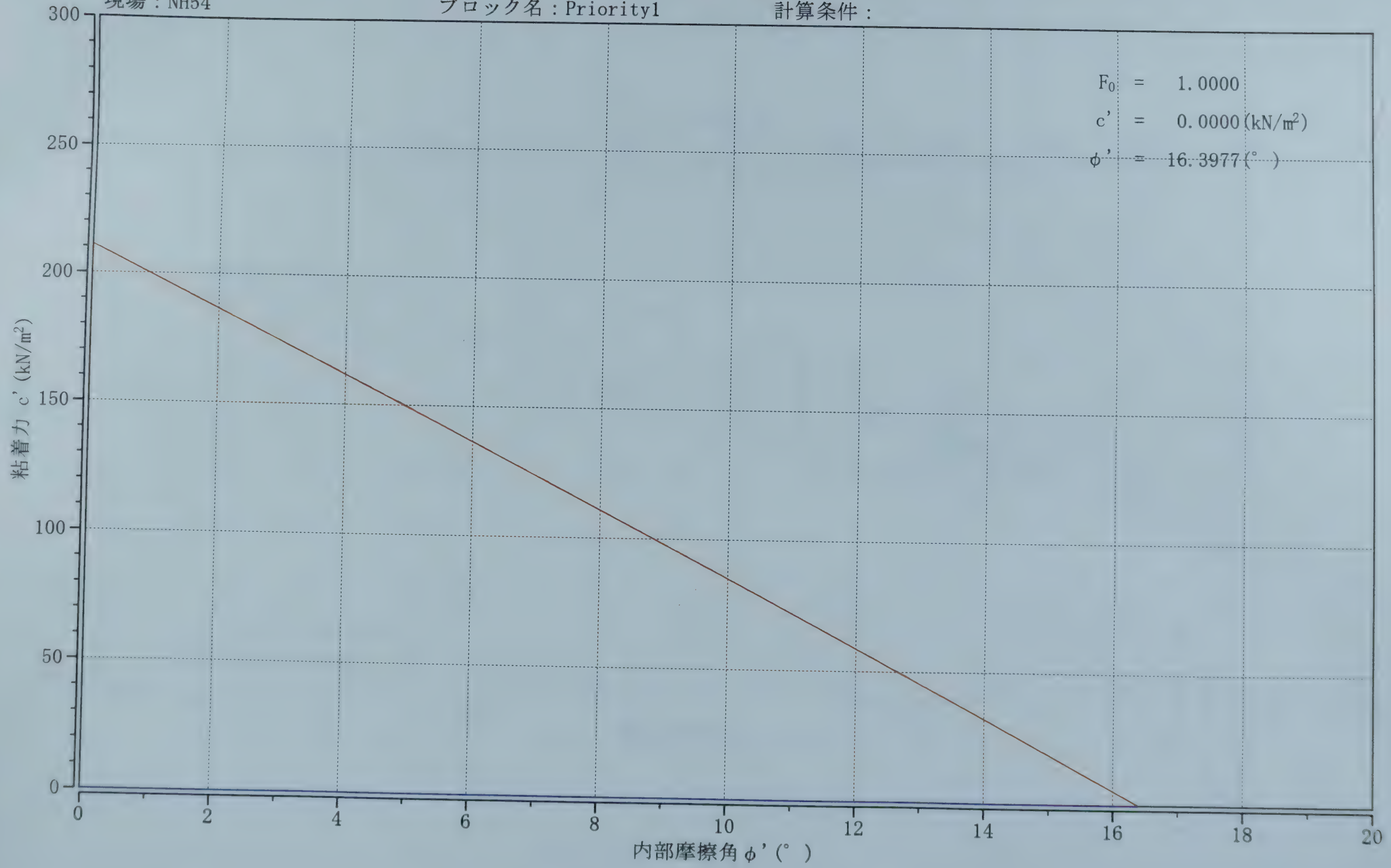


A03-59

c’－φ’図

現場：NH54　　ブロック名：Priority1　　計算条件：

F_0 = 1.0000
c’ = 0.0000 (kN/m²)
φ’ = 16.3977 (°)



A03-60

NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Previous	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m^3)	23.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)				杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000			杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	43.53	クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	513.54	水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0439	陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧		E_a (kN/m)	
移動層の断面積	A (m^2)	20989.39	二次元安全率		F_2	1.0000
地すべり抵抗力	Σ S (kN/m)	119364.67	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	Σ T (kN/m)	124606.79				

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	52.910	48.330	20.00	20.460	23285.20
2	10.350	13.230	7.22	4.910	1957.85	37	48.330	46.000	10.00	20.460	10847.95
3	13.230	24.710	8.44	4.910	3682.46	38	46.000	40.670	6.89	42.340	6867.30
4	24.710	27.310	2.23	4.910	1334.05	39	40.670	38.630	2.76	42.340	2516.98
5	27.310	27.870	12.96	4.910	8224.03	40	38.630	34.380	5.49	42.340	4609.49
6	27.870	28.740	10.00	4.910	6510.15	41	34.380	30.100	5.49	42.340	4070.94
7	28.740	33.260	12.60	4.910	8983.80	42	30.100	26.000	5.49	42.340	3541.87
8	33.260	39.530	8.43	4.910	7056.63	43	26.000	21.790	5.49	42.340	3017.22
9	39.530	40.900	8.98	4.910	8306.01	44	21.790	17.560	5.49	42.340	2484.36
10	40.900	42.610	13.36	4.910	12830.48	45	17.560	13.300	5.49	42.340	1948.35
11	42.610	41.170	16.88	4.910	16263.37	46	13.300	0.000	17.49	42.340	2675.10
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	43.200	6.60	4.910	6477.31						
15	43.200	46.320	19.59	4.910	20167.51						
16	46.320	49.510	20.00	4.910	22040.90						
17	49.510	52.710	20.00	4.910	23510.60						
18	52.710	52.710	20.00	13.790	24246.60						
19	52.710	52.710	20.00	13.790	24246.60						
20	52.710	52.710	8.02	13.790	9722.89						
21	52.710	52.710	7.96	13.790	9650.15						
22	52.710	52.710	21.88	13.790	26525.78						
23	52.710	52.710	2.14	13.790	2594.39						
24	52.710	52.710	23.63	13.790	28647.36						
25	52.710	52.710	16.37	13.790	19845.84						
26	52.710	52.710	13.94	13.790	16899.88						
27	52.710	52.710	8.12	13.790	9844.12						
28	52.710	52.710	5.97	13.790	7237.61						
29	52.710	52.710	1.97	13.790	2388.29						
30	52.710	51.990	5.59	20.460	6730.64						
31	51.990	51.840	1.18	20.460	1408.97						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	56.260	6.83	20.460	8645.45						
34	56.260	57.870	9.09	20.460	11930.58						
35	57.870	52.910	21.13	20.460	26918.99						

備考：

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	Previous	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

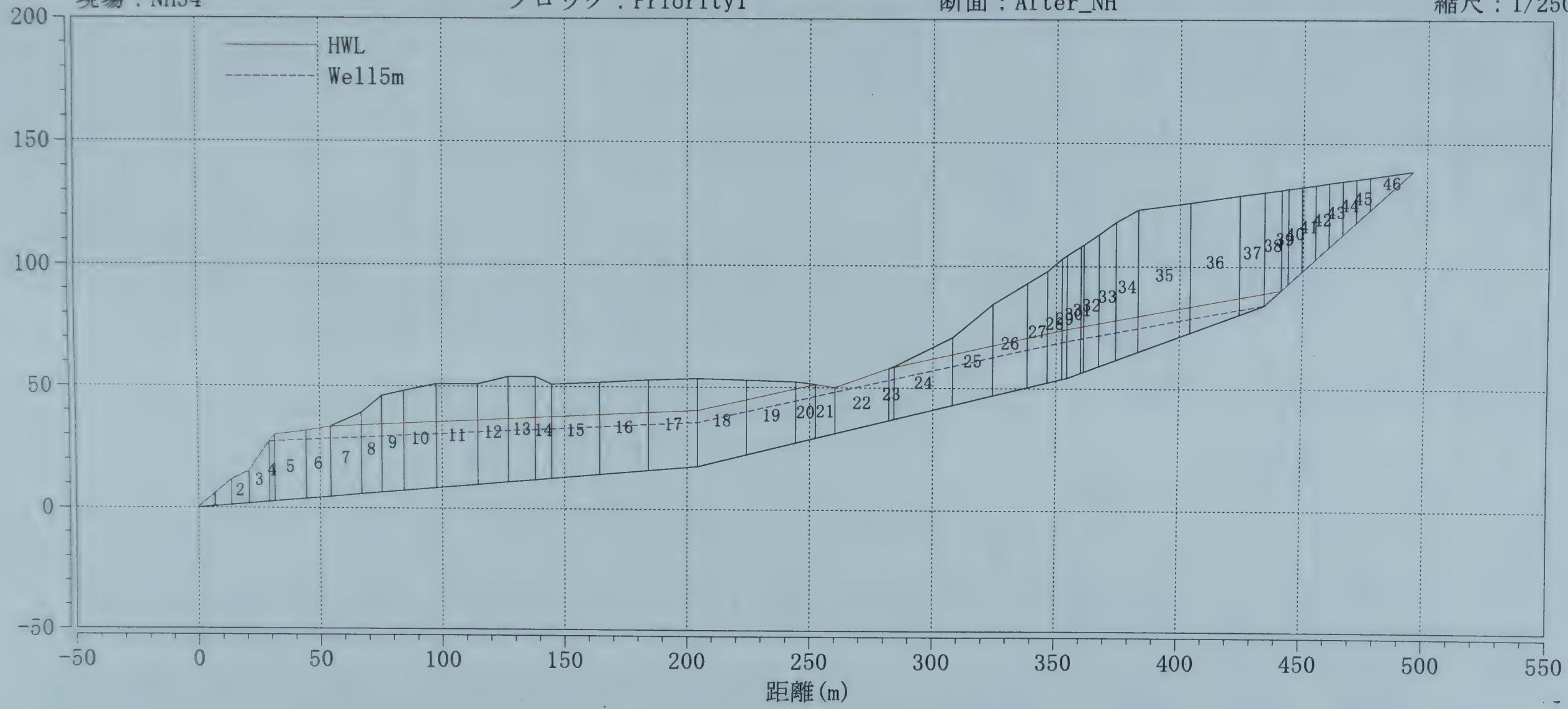
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000	設計済み負担推力	分子側	アンカー P_{sd} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)				杭 P_{gp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ud} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000			杭 P_{up} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{ux} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	119364.67	移動層の断面積		A (m^2)	20989.39
地すべり力	Σ T (kN/m)	124606.79	二次元安全率		F_2	1.0000

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0178	265.51	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0178	324.72	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0178	610.75	316.35
4	27.310	2.23				568.8	2.24			225.19	1.0178	221.26	114.60
5	27.870	12.96				3506.5	13.01			1388.23	1.0178	1363.99	706.49
6	28.740	10.00				2775.8	10.04			1098.92	1.0178	1079.74	559.26
7	28.260	12.60				3521.6	12.65			1607.38	1.0178	1579.32	771.76
8	27.940	8.43				2323.0	8.46			1392.97	1.0178	1368.65	606.21
9	27.590	8.98				2445.1	9.01			1724.71	1.0178	1694.60	713.54
10	27.080	13.36				3581.3	13.41			2721.76	1.0178	2674.25	1102.22
11	26.440	16.88				4429.8	16.94			3482.31	1.0178	3421.51	1397.12
12	25.960	12.53				3219.4	12.58			2625.52	1.0178	2579.68	1043.02
13	25.530	11.03				2784.8	11.07			2361.90	1.0178	2320.66	928.73
14	25.280	6.60				1644.3	6.62			1422.22	1.0178	1397.39	556.44
15	24.530	19.59				4784.6	19.66			4526.78	1.0178	4447.75	1732.51
16	23.770	20.00				4736.6	20.07			5092.17	1.0178	5003.27	1893.45
17	23.000	20.00				4586.6	20.07			5568.82	1.0178	5471.60	2019.70
18	22.600	20.00				4471.8	20.59			5819.17	1.0113	5754.12	5951.05
19	22.200	20.00				4393.4	20.59			5842.25	1.0113	5776.95	5951.05
20	22.040	8.02				1739.7	8.26			2349.22	1.0113	2322.96	2386.37
21	21.880	7.96				1714.2	8.20			2335.32	1.0113	2309.22	2368.52
22	21.440	21.88				4647.6	22.53			6438.15	1.0113	6366.18	6510.45
23	21.400	2.14				449.5	2.20			631.17	1.0113	624.12	636.76
24	20.930	23.63				4904.6	24.33			6986.84	1.0113	6908.74	7031.16
25	20.600	16.37				3333.5	16.86			4859.12	1.0113	4804.81	4870.93
26	20.320	13.94				2797.0	14.35			4150.09	1.0113	4103.70	4147.88
27	20.160	8.12				1611.7	8.36			2422.57	1.0113	2395.49	2416.13
28	20.040	5.97				1176.8	6.15			1783.54	1.0113	1763.60	1776.39
29	20.000	1.97				386.8	2.03			588.99	1.0113	582.41	586.18
30	18.950	5.59				1067.6	5.97			1666.48	0.9742	1710.63	2511.13
31	18.730	1.18				218.0	1.26			350.47	0.9742	359.75	525.67
32	17.570	6.18				1100.0	6.60			1885.86	0.9742	1935.83	2801.36
33	16.290	6.83				1134.0	7.29			2210.42	0.9742	2268.99	3225.52
34	14.590	9.09				1376.4	9.70			3105.82	0.9742	3188.11	4451.17
35	10.620	21.13				2611.9	22.55			7152.89	0.9742	7342.42	10043.18
36	6.870	20.00				1715.2	21.35			6347.46	0.9742	6515.65	8687.46
37	5.000	10.00				582.0	10.67			3020.98	0.9742	3101.02	4047.25
38	0.000	6.89				168.9	9.32			1971.15	0.6929	2844.94	6257.53
39						0.0	3.73			740.68	0.6929	1069.01	2293.49
40						0.0	7.43			1356.44	0.6929	1957.74	4200.20
41						0.0	7.43			1197.97	0.6929	1729.01	3709.47
42						0.0	7.43			1042.28	0.6929	1504.31	3227.38
43						0.0	7.43			887.89	0.6929	1281.48	2749.31
44						0.0	7.43			731.08	0.6929	1055.16	2263.77
45						0.0	7.43			573.34	0.6929	827.50	1775.35
46						0.0	23.66			787.21	0.6929	1136.17	2437.57

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：After_NH　　縮尺：1/2500



NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	After_NH	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m³)	23.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)
移動層の水中単位体積重量	γ' (kN/m³)				杭 P_{sp} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他 P_{sx} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭 P_{mp} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{mx} (kN/m)
弦～すべり面の最大深さ	d (m)	43.53	クラックの水位高		Z_w (m)
地すべり斜面弦長	L (m)	513.54	水圧の水平成分		Q_w (kN/m)
修正係数	f_0	1.0439	陥没帯下流壁のすべり面深さ		Z_h (m)
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数		k_a
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧		E_a (kN/m)
移動層の断面積	A (m²)	16900.18	二次元安全率		F_2 0.9195
地すべり抵抗力	ΣS (kN/m)	91489.56	$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$		
地すべり力	ΣT (kN/m)	103869.03			

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	52.910	48.330	20.00	20.460	23285.20
2	10.350	13.230	7.22	4.910	1957.85	37	48.330	46.000	10.00	20.460	10847.95
3	13.230	24.710	8.44	4.910	3682.46	38	46.000	40.670	6.89	42.340	6867.30
4	24.710	27.310	2.23	4.910	1334.05	39	40.670	38.630	2.76	42.340	2516.98
5	27.310	27.870	12.96	4.910	8224.03	40	38.630	34.380	5.49	42.340	4609.49
6	27.870	28.740	10.00	4.910	6510.15	41	34.380	30.100	5.49	42.340	4070.94
7	28.740	33.260	12.60	4.910	8983.80	42	30.100	26.000	5.49	42.340	3541.87
8	33.260	39.530	8.43	4.910	7056.63	43	26.000	21.790	5.49	42.340	3017.22
9	39.530	40.900	8.98	4.910	8306.01	44	21.790	17.560	5.49	42.340	2484.36
10	40.900	42.610	13.36	4.910	12830.48	45	17.560	13.300	5.49	42.340	1948.35
11	42.610	41.170	16.88	4.910	16263.37	46	13.300	0.000	17.49	42.340	2675.10
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.840	1.18	20.460	1403.14						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	56.260	6.83	20.460	8645.45						
34	56.260	57.870	9.09	20.460	11930.58						
35	57.870	52.910	21.13	20.460	26918.99						

備考：

NH54_priority 1
st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	After_NH	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_c (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	91489.56		移動層の断面積	A (m^2)	16900.18	
地すべり力	Σ T (kN/m)	103869.03		二次元安全率	F_2	0.9195	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 S=s_l / n_α (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0200	264.94	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0200	324.02	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0200	609.44	316.35
4	27.310	2.23				568.8	2.24			225.19	1.0200	220.78	114.60
5	27.870	12.96				3506.5	13.01			1388.23	1.0200	1361.06	706.49
6	28.740	10.00				2775.8	10.04			1098.92	1.0200	1077.41	559.26
7	28.260	12.60				3521.6	12.65			1607.38	1.0200	1575.92	771.76
8	27.940	8.43				2323.0	8.46			1392.97	1.0200	1365.70	606.21
9	27.590	8.98				2445.1	9.01			1724.71	1.0200	1690.95	713.54
10	27.080	13.36				3581.3	13.41			2721.76	1.0200	2668.49	1102.22
11	26.440	16.88				4429.8	16.94			3482.31	1.0200	3414.14	1397.12
12	25.960	12.53				3219.4	12.58			2625.52	1.0200	2574.12	1043.02
13	25.530	11.03				2784.8	11.07			2361.90	1.0200	2315.66	928.73
14	25.280	6.60				1644.3	6.62			1319.03	1.0200	1293.21	526.32
15	24.530	19.59				4784.6	19.66			3651.68	1.0200	3580.20	1477.05
16	23.770	20.00				4736.6	20.07			3668.13	1.0200	3596.33	1477.73
17	23.000	20.00				4586.6	20.07			3594.52	1.0200	3524.15	1443.35
18	22.600	20.00				4471.8	20.59			3188.34	1.0173	3134.22	3756.80
19	22.200	20.00				4393.4	20.59			2464.89	1.0173	2423.05	3134.15
20	22.040	8.02				1739.7	8.26			763.93	1.0173	750.97	1064.16
21	18.890	7.96				1597.5	8.20			632.45	1.0173	621.72	919.59
22	21.440	21.88				4326.8	22.53			1712.97	1.0173	1683.89	2490.67
23	21.400	2.14				449.5	2.20			179.78	1.0173	176.73	260.27
24	20.930	23.63				4904.6	24.33			2511.09	1.0173	2468.46	3298.15
25	20.600	16.37				3333.5	16.86			2629.90	1.0173	2585.26	3011.64
26	20.320	13.94				2797.0	14.35			2949.50	1.0173	2899.43	3146.52
27	20.160	8.12				1611.7	8.36			1948.83	1.0173	1915.75	2021.00
28	20.040	5.97				1176.8	6.15			1567.36	1.0173	1540.76	1596.09
29	20.000	1.97				386.8	2.03			547.06	1.0173	537.77	551.20
30	18.950	5.59				1067.6	5.97			1607.07	0.9826	1635.49	2435.82
31	18.730	1.18				218.0	1.26			348.75	0.9826	354.92	523.50
32	17.570	6.18				1100.0	6.60			1885.86	0.9826	1919.21	2801.36
33	16.290	6.83				1134.0	7.29			2210.42	0.9826	2249.51	3225.52
34	14.590	9.09				1376.4	9.70			3105.82	0.9826	3160.74	4451.17
35	10.620	21.13				2611.9	22.55			7152.89	0.9826	7279.38	10043.18
36	6.870	20.00				1715.2	21.35			6347.46	0.9826	6459.70	8687.46
37	5.000	10.00				582.0	10.67			3020.98	0.9826	3074.40	4047.25
38	0.000	6.89				168.9	9.32			1971.15	0.7057	2793.23	6257.53
39						0.0	3.73			740.68	0.7057	1049.59	2293.49
40						0.0	7.43			1356.44	0.7057	1922.16	4200.20
41						0.0	7.43			1197.97	0.7057	1697.59	3709.47
42						0.0	7.43			1042.28	0.7057	1476.97	3227.38
43						0.0	7.43			887.89	0.7057	1258.19	2749.31
44						0.0	7.43			731.08	0.7057	1035.98	2263.77
45						0.0	7.43			573.34	0.7057	812.46	1775.35
46						0.0	23.66			787.21	0.7057	1115.52	2437.57

備考：

NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	After_NH	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m^3)	23.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)				杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000			杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	43.53	クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	513.54	水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0439	陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧		E_a (kN/m)	
移動層の断面積	A (m^2)	16900.18	二次元安全率		F_2	0.9814
地すべり抵抗力	Σ S (kN/m)	97645.11	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	Σ T (kN/m)	103869.03				

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	52.910	48.330	20.00	20.460	23285.20
2	10.350	13.230	7.22	4.910	1957.85	37	48.330	46.000	10.00	20.460	10847.95
3	13.230	24.710	8.44	4.910	3682.46	38	46.000	40.670	6.89	42.340	6867.30
4	24.710	27.310	2.23	4.910	1334.05	39	40.670	38.630	2.76	42.340	2516.98
5	27.310	27.870	12.96	4.910	8224.03	40	38.630	34.380	5.49	42.340	4609.49
6	27.870	28.740	10.00	4.910	6510.15	41	34.380	30.100	5.49	42.340	4070.94
7	28.740	33.260	12.60	4.910	8983.80	42	30.100	26.000	5.49	42.340	3541.87
8	33.260	39.530	8.43	4.910	7056.63	43	26.000	21.790	5.49	42.340	3017.22
9	39.530	40.900	8.98	4.910	8306.01	44	21.790	17.560	5.49	42.340	2484.36
10	40.900	42.610	13.36	4.910	12830.48	45	17.560	13.300	5.49	42.340	1948.35
11	42.610	41.170	16.88	4.910	16263.37	46	13.300	0.000	17.49	42.340	2675.10
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.840	1.18	20.460	1403.14						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	56.260	6.83	20.460	8645.45						
34	56.260	57.870	9.09	20.460	11930.58						
35	57.870	52.910	21.13	20.460	26918.99						

備考：

NH54_priority 1
st.WS1

二　次　元　安　定　解　析　表

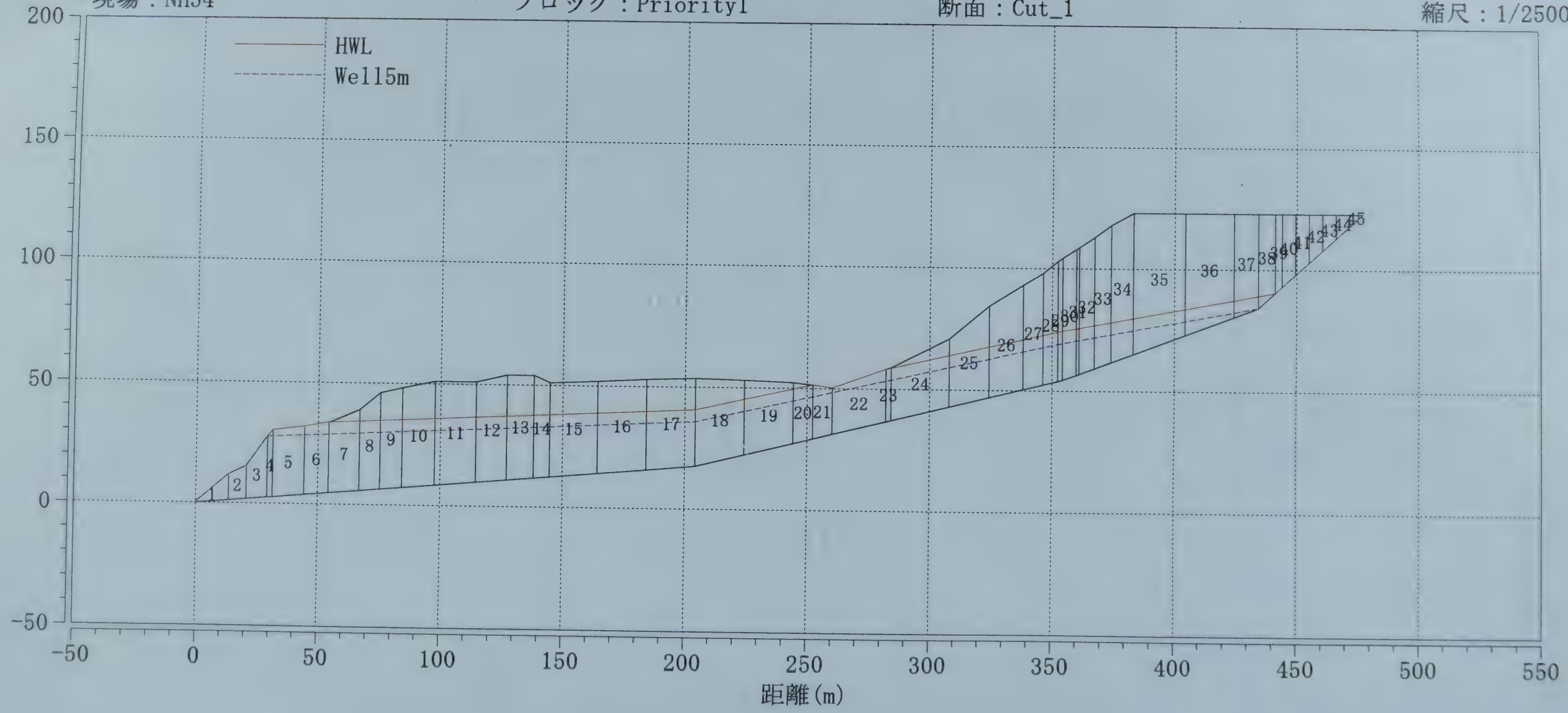
現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	After_NH	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x=　　, y=　　, z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000		設計済み負担推力 分子側	アンカー	P_{sd} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)				杭	P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)				その他	P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー	P_{md} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000			杭	P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他	P_{mx} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	97645.11		移動層の断面積	A (m^2)	16900.18	
地すべり力	Σ T (kN/m)	103869.03		二次元安全率	F_2	0.9814	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0182	265.39	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0182	324.57	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0182	610.47	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0182	229.66	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0182	1481.63	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0182	1203.23	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0182	1757.13	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0182	1487.47	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0182	1821.05	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0182	2862.31	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0182	3659.11	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0182	2756.03	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0182	2475.88	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0182	1388.92	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0182	3863.85	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0182	3885.82	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0182	3813.52	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0126	3433.67	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0126	2719.22	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0126	868.71	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0126	704.10	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0126	1910.22	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0126	208.03	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0126	2816.56	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0126	2830.45	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0126	3111.44	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0126	2040.29	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0126	1632.93	1596.09
29	15.000	1.97				290.2	2.03			575.48	1.0126	568.32	551.20
30	13.950	5.59				793.5	5.97			1687.73	0.9760	1729.21	2435.82
31	13.730	1.18				160.2	1.26			365.78	0.9760	374.76	523.50
32	12.570	6.18				797.0	6.60			1975.04	0.9760	2023.57	2801.36
33	11.290	6.83				799.1	7.29			2308.97	0.9760	2365.71	3225.52
34	9.590	9.09				930.6	9.70			3236.98	0.9760	3316.52	4451.17
35	5.630	21.13				1576.9	22.55			7457.48	0.9760	7640.73	10043.18
36	1.880	20.00				736.5	21.35			6635.47	0.9760	6798.52	8687.46
37	0.000	10.00				92.2	10.67			3165.13	0.9760	3242.90	4047.25
38						0.0	9.32			2020.86	0.6956	2905.02	6257.53
39						0.0	3.73			740.68	0.6956	1064.74	2293.49
40						0.0	7.43			1356.44	0.6956	1949.92	4200.20
41						0.0	7.43			1197.97	0.6956	1722.10	3709.47
42						0.0	7.43			1042.28	0.6956	1498.29	3227.38
43						0.0	7.43			887.89	0.6956	1276.35	2749.31
44						0.0	7.43			731.08	0.6956	1050.94	2263.77
45						0.0	7.43			573.34	0.6956	824.20	1775.35
46						0.0	23.66			787.21	0.6956	1131.63	2437.57

備考

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_1　　縮尺：1/2500



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NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_1	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000		設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)
移動層の水中単位体積重量	γ' (kN/m³)					杭	P_{sp} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m³)					その他	P_{sx} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー	P_{ma} (kN/m)
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭	P_{mp} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他	P_{mx} (kN/m)
弦～すべり面の最大深さ	d (m)	35.86		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	492.50		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0392		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	16163.57		二次元安全率		F_2	1.0475
地すべり抵抗力	ΣS (kN/m)	91420.92		$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	ΣT (kN/m)	90696.89					

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	49.990	42.530	20.00	20.460	21279.60
2	10.350	13.230	7.22	4.910	1957.85	37	42.530	38.800	10.00	20.460	9352.95
3	13.230	24.710	8.44	4.910	3682.46	38	38.800	32.520	6.89	42.340	5651.04
4	24.710	27.310	2.23	4.910	1334.05	39	32.520	30.000	2.76	42.340	1984.38
5	27.310	27.870	12.96	4.910	8224.03	40	30.000	25.000	5.49	42.340	3472.43
6	27.870	28.740	10.00	4.910	6510.15	41	25.000	20.000	5.49	42.340	2841.08
7	28.740	33.260	12.60	4.910	8983.80	42	20.000	15.000	5.49	42.340	2209.73
8	33.260	39.530	8.43	4.910	7056.63	43	15.000	10.000	5.49	42.340	1578.38
9	39.530	40.900	8.98	4.910	8306.01	44	10.000	5.000	5.49	42.340	947.03
10	40.900	42.610	13.36	4.910	12830.48	45	5.000	0.000	5.49	42.340	315.68
11	42.610	41.170	16.88	4.910	16263.37						
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.840	1.18	20.460	1403.14						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	56.260	6.83	20.460	8645.45						
34	56.260	57.870	9.09	20.460	11930.58						
35	57.870	49.990	21.13	20.460	26209.44						

備考：

二 次 元 安 定 解 析 表

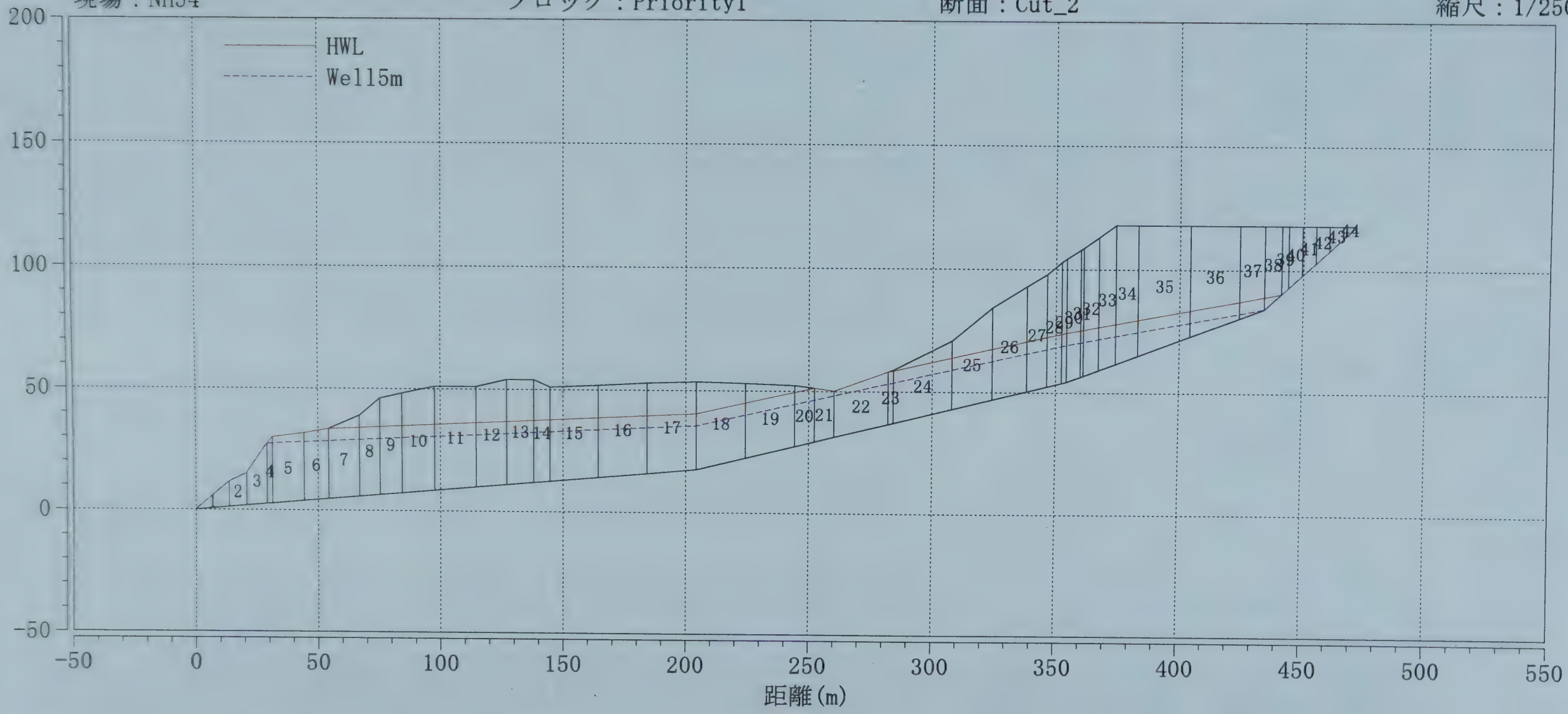
現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下端撃 k_0	
断面名	Cut_1	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	91420.92		移動層の断面積	A (m^2)	16163.57	
地すべり力	Σ T (kN/m)	90696.89		二次元安全率	F_2	1.0475	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_l / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0166	265.81	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0166	325.08	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0166	611.44	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0166	230.02	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0166	1483.98	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0166	1205.14	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0166	1759.92	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0166	1489.83	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0166	1823.95	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0166	2866.86	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0166	3664.92	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0166	2760.40	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0166	2479.81	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0166	1391.12	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0166	3869.99	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0166	3891.99	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0166	3819.58	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0082	3448.59	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0082	2731.04	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0082	872.49	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0082	707.16	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0082	1918.52	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0082	208.94	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0082	2828.80	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0082	2842.75	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0082	3124.96	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0082	2049.16	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0082	1640.03	1596.09
29	15.000	1.97				290.2	2.03			575.48	1.0082	570.79	551.20
30	13.950	5.59				793.5	5.97			1687.73	0.9698	1740.26	2435.82
31	13.730	1.18				160.2	1.26			365.78	0.9698	377.16	523.50
32	12.570	6.18				797.0	6.60			1975.04	0.9698	2036.50	2801.36
33	11.290	6.83				799.1	7.29			2308.97	0.9698	2380.83	3225.52
34	9.590	9.09				930.6	9.70			3236.98	0.9698	3337.72	4451.17
35	5.630	21.13				1576.9	22.55			7248.68	0.9698	7474.27	9778.46
36	1.880	20.00				736.5	21.35			6045.27	0.9698	6233.41	7939.19
37	0.000	10.00				92.2	10.67			2725.19	0.9698	2810.00	3489.48
38						0.0	9.32			1662.95	0.6862	2423.35	5149.27
39						0.0	3.73			583.95	0.6862	850.97	1808.19
40						0.0	7.43			1021.84	0.6862	1489.09	3164.10
41						0.0	7.43			836.05	0.6862	1218.34	2588.81
42						0.0	7.43			650.26	0.6862	947.60	2013.52
43						0.0	7.43			464.47	0.6862	676.86	1438.23
44						0.0	7.43			278.68	0.6862	406.11	862.94
45						0.0	7.43			92.89	0.6862	135.37	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_2　　縮尺：1/2500



A03-71

NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_2	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000	設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)				杭	P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他	P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー	P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭	P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他	P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	33.31	クラックの水位高			Z_w (m)	
地すべり斜面弦長	L (m)	485.95	水圧の水平成分			Q_w (kN/m)	
修正係数	f_0	1.0373	陥没帯下流壁のすべり面深さ			Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数			k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧			E_a (kN/m)	
移動層の断面積	A (m²)	15685.97	二次元安全率			F_2	1.0813
地すべり抵抗力	ΣS (kN/m)	87696.81	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$				
地すべり力	ΣT (kN/m)	84132.83					

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	44.990	37.530	20.00	20.460	18979.60
2	10.350	13.230	7.22	4.910	1957.85	37	37.530	33.800	10.00	20.460	8202.95
3	13.230	24.710	8.44	4.910	3682.46	38	33.800	27.520	6.89	42.340	4858.69
4	24.710	27.310	2.23	4.910	1334.05	39	27.520	25.000	2.76	42.340	1666.98
5	27.310	27.870	12.96	4.910	8224.03	40	25.000	20.000	5.49	42.340	2841.08
6	27.870	28.740	10.00	4.910	6510.15	41	20.000	15.000	5.49	42.340	2209.73
7	28.740	33.260	12.60	4.910	8983.80	42	15.000	10.000	5.49	42.340	1578.38
8	33.260	39.530	8.43	4.910	7056.63	43	10.000	5.000	5.49	42.340	947.03
9	39.530	40.900	8.98	4.910	8306.01	44	5.000	0.000	5.49	42.340	315.68
10	40.900	42.610	13.36	4.910	12830.48						
11	42.610	41.170	16.88	4.910	16263.37						
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.840	1.18	20.460	1403.14						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	56.260	6.83	20.460	8645.45						
34	56.260	52.870	9.09	20.460	11407.90						
35	52.870	44.990	21.13	20.460	23779.49						

備考：

NH54_priority 1
st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1
断面名	Cut_2	間隙水圧名	Well5m
計算条件			

安定解析式	簡易Janbu法	陥没帯下流壁 No	
地下水条件	被圧地下水		
水平外力 Q(kN/m)	0.00	水平震度 k_h	
湛水条件	なし	基準水面の標高BL(m)	
末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	16.3977
地すべり抵抗力	Σ S (kN/m)	87696.81
地すべり力	Σ T (kN/m)	84132.83

設計済み負担推力			
分子側	アンカー	P_{sa} (kN/m)	
	杭	P_{sp} (kN/m)	
	その他	P_{sx} (kN/m)	
分母側	アンカー	P_{ma} (kN/m)	
	杭	P_{mp} (kN/m)	
	その他	P_{mx} (kN/m)	

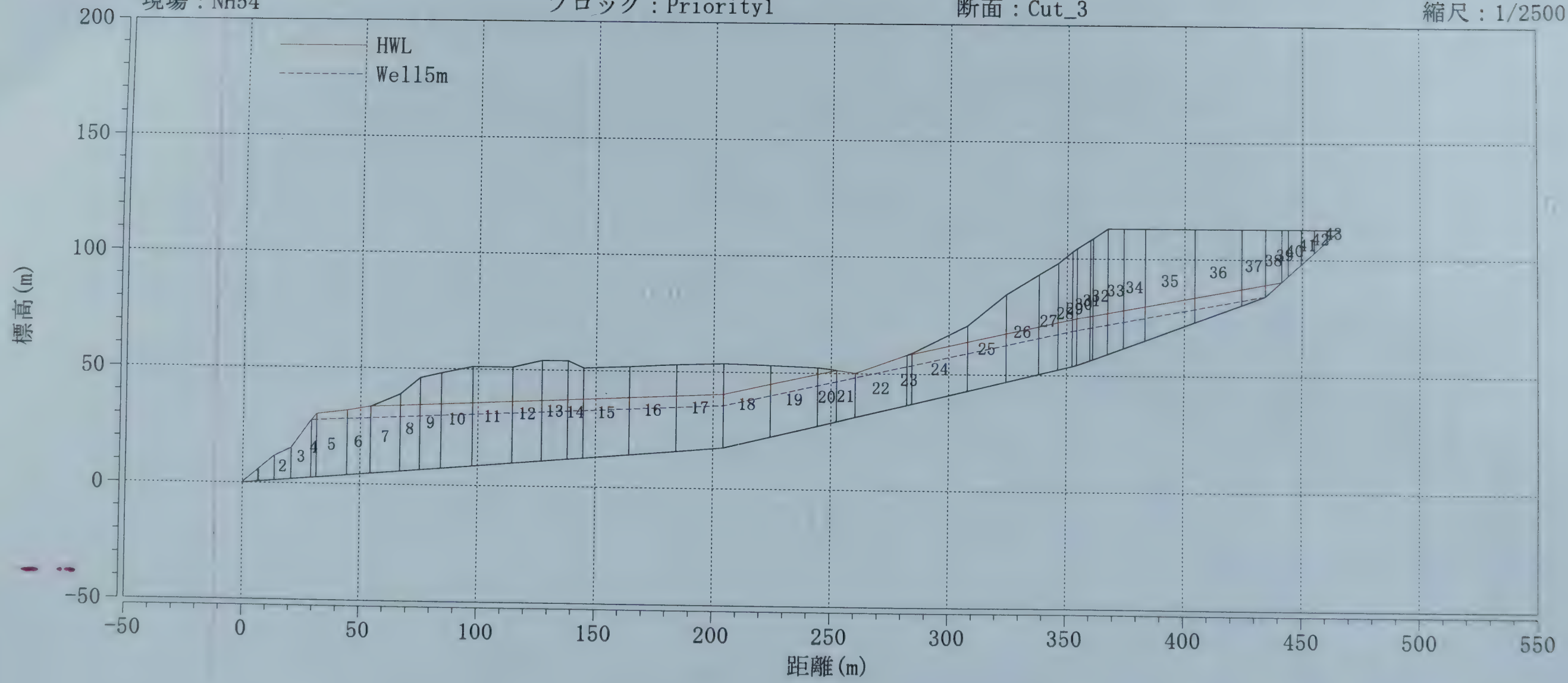
移動層の断面積	A (m^2)	15685.97
二次元安全率	F_2	1.0813

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 S=s_1/n_α (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0159	266.01	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0159	325.32	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0159	611.89	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0159	230.19	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0159	1485.07	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0159	1206.02	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0159	1761.22	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0159	1490.93	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0159	1825.29	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0159	2868.97	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0159	3667.62	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0159	2762.44	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0159	2481.64	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0159	1392.15	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0159	3872.84	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0159	3894.85	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0159	3822.39	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0062	3455.56	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0062	2736.55	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0062	874.25	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0062	708.59	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0062	1922.40	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0062	209.36	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0062	2834.52	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0062	2848.49	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0062	3131.27	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0062	2053.30	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0062	1643.34	1596.09
29	15.000	1.97				290.2	2.03			575.48	1.0062	571.95	551.20
30	13.950	5.59				793.5	5.97			1687.73	0.9669	1745.43	2435.82
31	13.730	1.18				160.2	1.26			365.78	0.9669	378.28	523.50
32	12.570	6.18				797.0	6.60			1975.04	0.9669	2042.56	2801.36
33	11.290	6.83				799.1	7.29			2308.97	0.9669	2387.91	3225.52
34	9.590	9.09				930.6	9.70			3083.17	0.9669	3188.57	4256.17
35	5.630	21.13				1576.9	22.55			6533.61	0.9669	6756.98	8871.87
36	1.880	20.00				736.5	21.35			5368.45	0.9669	5551.98	7081.08
37	0.000	10.00				92.2	10.67			2386.77	0.9669	2468.37	3060.43
38						0.0	9.32			1429.78	0.6818	2096.91	4427.27
39						0.0	3.73			490.55	0.6818	719.44	1518.97
40						0.0	7.43			836.05	0.6818	1226.15	2588.81
41						0.0	7.43			650.26	0.6818	953.67	2013.52
42						0.0	7.43			464.47	0.6818	681.20	1438.23
43						0.0	7.43			278.68	0.6818	408.72	862.94
44						0.0	7.43			92.89	0.6818	136.24	287.65

備考

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_3　　縮尺：1/2500



A03-74

NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_3	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m³)	23.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)				杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	31.10	クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	479.42	水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0356	陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	15196.02	二次元安全率		F_2	1.1172
地すべり抵抗力	ΣS (kN/m)	83931.36	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	ΣT (kN/m)	77802.54				

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	39.990	32.530	20.00	20.460	16679.60
2	10.350	13.230	7.22	4.910	1957.85	37	32.530	28.800	10.00	20.460	7052.95
3	13.230	24.710	8.44	4.910	3682.46	38	28.800	22.520	6.89	42.340	4066.34
4	24.710	27.310	2.23	4.910	1334.05	39	22.520	20.000	2.76	42.340	1349.58
5	27.310	27.870	12.96	4.910	8224.03	40	20.000	15.000	5.49	42.340	2209.73
6	27.870	28.740	10.00	4.910	6510.15	41	15.000	10.000	5.49	42.340	1578.38
7	28.740	33.260	12.60	4.910	8983.80	42	10.000	5.000	5.49	42.340	947.03
8	33.260	39.530	8.43	4.910	7056.63	43	5.000	0.000	5.49	42.340	315.68
9	39.530	40.900	8.98	4.910	8306.01						
10	40.900	42.610	13.36	4.910	12830.48						
11	42.610	41.170	16.88	4.910	16263.37						
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.840	1.18	20.460	1403.14						
32	51.840	53.810	6.18	20.460	7508.55						
33	53.810	51.260	6.83	20.460	8252.72						
34	51.260	47.870	9.09	20.460	10362.55						
35	47.870	39.990	21.13	20.460	21349.54						

備考：

二 次 元 安 定 解 析 表

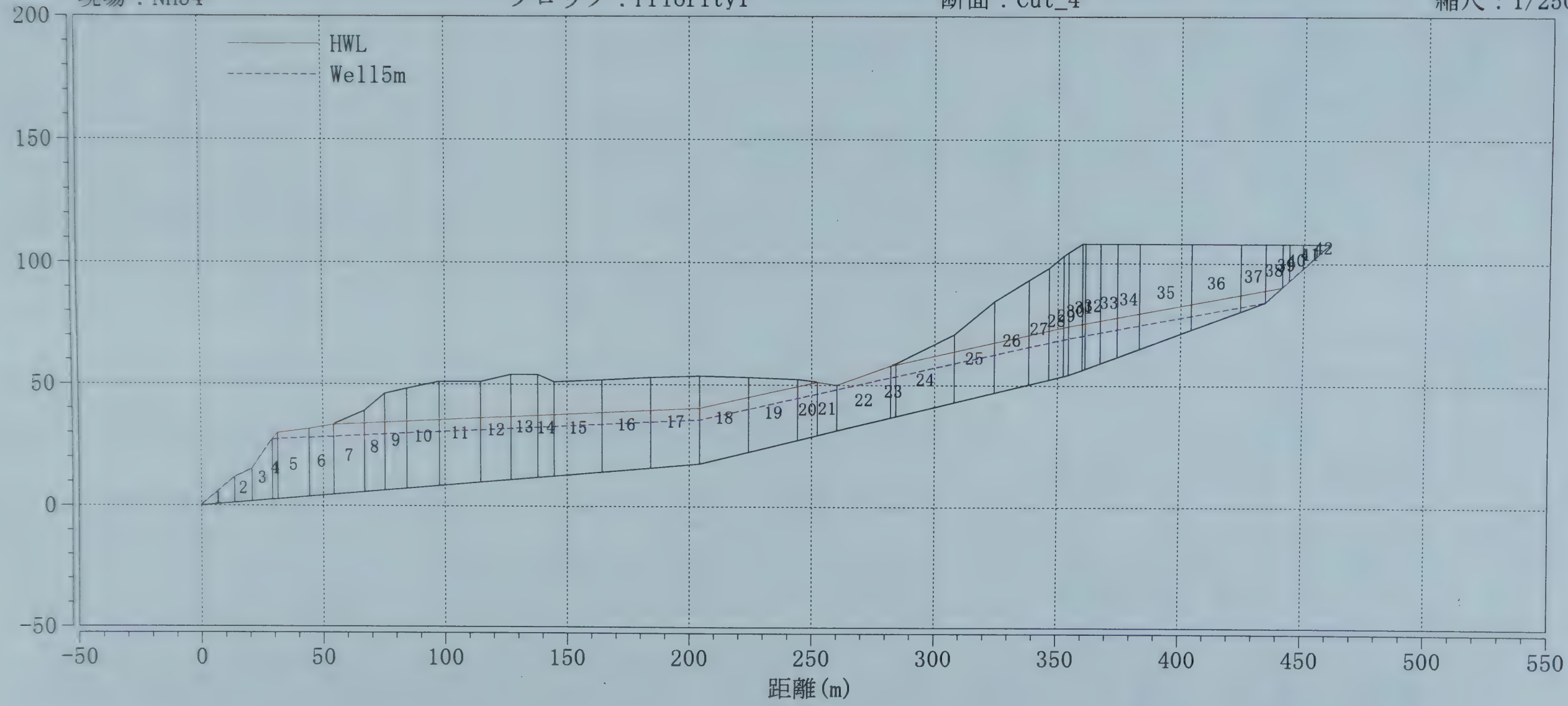
現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯上流撃 No	
断面名	Cut_3	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	83931.36		移動層の断面積	A (m^2)	15196.02	
地すべり力	Σ T (kN/m)	77802.54		二次元安全率	F_2	1.1172	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0151	266.20	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0151	325.56	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0151	612.34	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0151	230.36	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0151	1486.16	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0151	1206.91	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0151	1762.51	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0151	1492.02	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0151	1826.63	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0151	2871.08	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0151	3670.32	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0151	2764.47	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0151	2483.46	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0151	1393.17	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0151	3875.69	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0151	3897.72	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0151	3825.20	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0042	3462.53	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0042	2742.07	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0042	876.01	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0042	710.02	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0042	1926.28	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0042	209.78	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0042	2840.24	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0042	2854.24	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0042	3137.59	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0042	2057.44	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0042	1646.66	1596.09
29	15.000	1.97				290.2	2.03			575.48	1.0042	573.10	551.20
30	13.950	5.59				793.5	5.97			1687.73	0.9641	1750.62	2435.82
31	13.730	1.18				160.2	1.26			365.78	0.9641	379.41	523.50
32	12.570	6.18				797.0	6.60			1975.04	0.9641	2048.63	2801.36
33	11.290	6.83				799.1	7.29			2193.41	0.9641	2275.14	3079.00
34	9.590	9.09				930.6	9.70			2775.55	0.9641	2878.98	3866.16
35	5.630	21.13				1576.9	22.55			5818.54	0.9641	6035.36	7965.28
36	1.880	20.00				736.5	21.35			4691.62	0.9641	4866.45	6222.98
37	0.000	10.00				92.2	10.67			2048.36	0.9641	2124.69	2631.38
38						0.0	9.32			1196.61	0.6775	1766.24	3705.28
39						0.0	3.73			397.15	0.6775	586.20	1229.75
40						0.0	7.43			650.26	0.6775	959.81	2013.52
41						0.0	7.43			464.47	0.6775	685.58	1438.23
42						0.0	7.43			278.68	0.6775	411.35	862.94
43						0.0	7.43			92.89	0.6775	137.12	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_4　　縮尺：1/2500



NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_4	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
				末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	16.3977
弦～すべり面の最大深さ	d (m)	29.58
地すべり斜面弦長	L (m)	472.92
修正係数	f_0	1.0345
冠頭部クラックの傾斜角	θ_u (°)	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)	
移動層の断面積	A (m^2)	14698.34
地すべり抵抗力	ΣS (kN/m)	80160.65
地すべり力	ΣT (kN/m)	71745.70

設計済み負担推力				
分子側	アンカー	P_{sa}	(kN/m)	
	杭	P_{sp}	(kN/m)	
	その他	P_{sx}	(kN/m)	
分母側	アンカー	P_{ma}	(kN/m)	
	杭	P_{mp}	(kN/m)	
	その他	P_{mx}	(kN/m)	

クラックの水位高	Z_w	(m)	
水圧の水平成分	Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ	Z_h	(m)	
主働土圧係数	k_a		
主働土圧	E_a	(kN/m)	
二次元安全率	F_2		1.1558

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	34.990	27.530	20.00	20.460	14379.60
2	10.350	13.230	7.22	4.910	1957.85	37	27.530	23.800	10.00	20.460	5902.95
3	13.230	24.710	8.44	4.910	3682.46	38	23.800	17.520	6.89	42.340	3273.99
4	24.710	27.310	2.23	4.910	1334.05	39	17.520	15.000	2.76	42.340	1032.18
5	27.310	27.870	12.96	4.910	8224.03	40	15.000	10.000	5.49	42.340	1578.38
6	27.870	28.740	10.00	4.910	6510.15	41	10.000	5.000	5.49	42.340	947.03
7	28.740	33.260	12.60	4.910	8983.80	42	5.000	0.000	5.49	42.340	315.68
8	33.260	39.530	8.43	4.910	7056.63						
9	39.530	40.900	8.98	4.910	8306.01						
10	40.900	42.610	13.36	4.910	12830.48						
11	42.610	41.170	16.88	4.910	16263.37						
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	50.000	1.97	13.790	2245.79						
30	50.000	51.560	5.59	20.460	6528.78						
31	51.560	51.120	1.18	20.460	1393.37						
32	51.120	48.810	6.18	20.460	7102.03						
33	48.810	46.260	6.83	20.460	7467.27						
34	46.260	42.870	9.09	20.460	9317.20						
35	42.870	34.990	21.13	20.460	18919.59						

備考：

NH54_priority 1
st.WS1

二 次 元 安 定 解 析 表

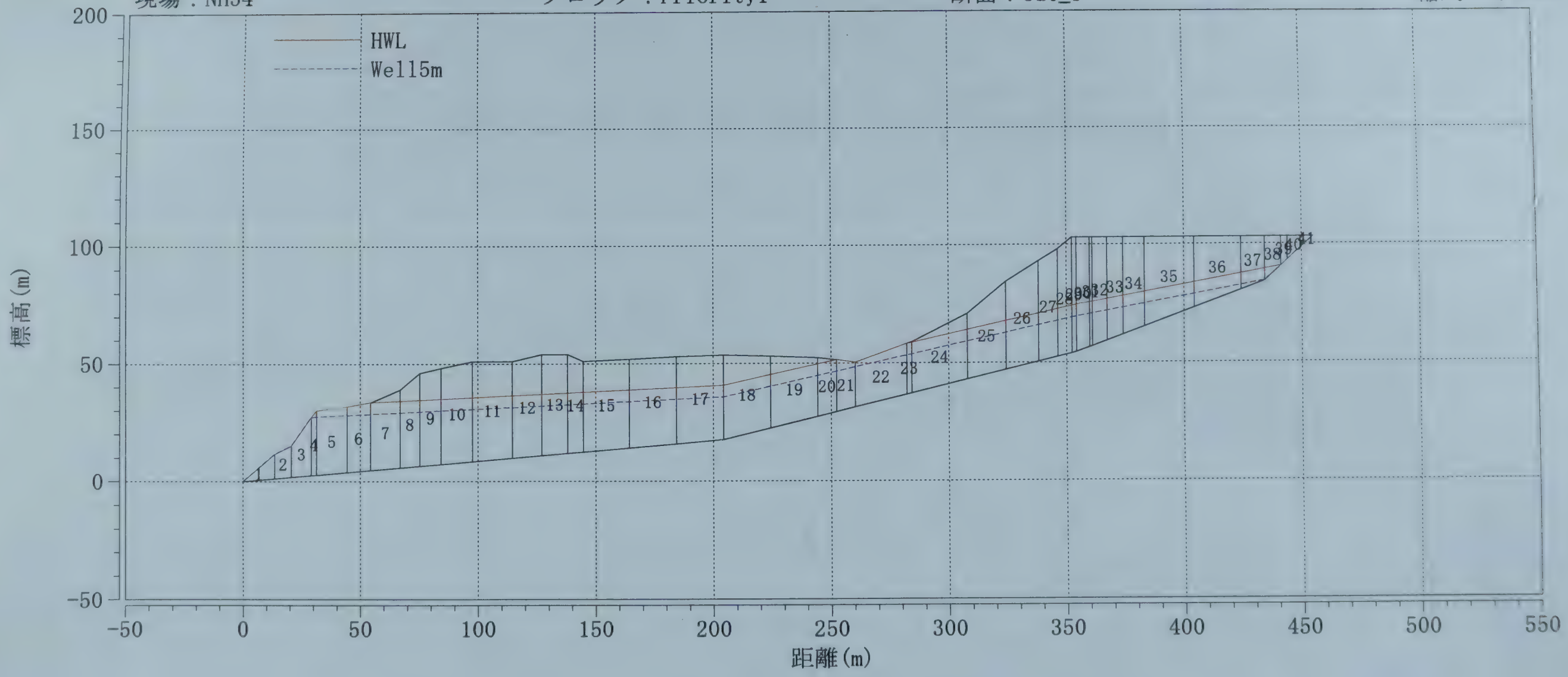
現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	Cut_4	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	80160.65		移動層の断面積	A (m^2)	14698.34	
地すべり力	Σ T (kN/m)	71745.70		二次元安全率	F_2	1.1558	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0144	266.40	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0144	325.80	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0144	612.79	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0144	230.53	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0144	1487.26	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0144	1207.80	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0144	1763.82	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0144	1493.13	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0144	1827.98	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0144	2873.21	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0144	3673.03	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0144	2766.52	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0144	2485.30	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0144	1394.20	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0144	3878.56	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0144	3900.60	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0144	3828.03	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0021	3469.57	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0021	2747.65	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0021	877.79	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0021	711.46	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0021	1930.19	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0021	210.21	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0021	2846.01	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0021	2860.04	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0021	3143.97	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0021	2061.62	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0021	1650.01	1596.09
29	15.000	1.97				290.2	2.03			575.48	1.0021	574.27	551.20
30	13.950	5.59				793.5	5.97			1687.73	0.9612	1755.87	2435.82
31	13.730	1.18				160.2	1.26			362.90	0.9612	377.55	519.85
32	12.570	6.18				797.0	6.60			1855.41	0.9612	1930.32	2649.69
33	11.290	6.83				799.1	7.29			1962.27	0.9612	2041.49	2785.96
34	9.590	9.09				930.6	9.70			2467.93	0.9612	2567.57	3476.15
35	5.630	21.13				1576.9	22.55			5103.47	0.9612	5309.52	7058.69
36	1.880	20.00				736.5	21.35			4014.79	0.9612	4176.89	5364.87
37	0.000	10.00				92.2	10.67			1709.95	0.9612	1778.99	2202.33
38						0.0	9.32			963.44	0.6731	1431.34	2983.28
39						0.0	3.73			303.74	0.6731	451.26	940.53
40						0.0	7.43			464.47	0.6731	690.04	1438.23
41						0.0	7.43			278.68	0.6731	414.02	862.94
42						0.0	7.43			92.89	0.6731	138.01	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_5　　縮尺：1/2500



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NH54_priority 1 st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_5	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m³)	23.000	設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)				杭	P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他	P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー	P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭	P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他	P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	28.00	クラックの水位高			Z_w (m)	
地すべり斜面弦長	L (m)	466.45	水圧の水平成分			Q_w (kN/m)	
修正係数	f_0	1.0333	陥没帯下流壁のすべり面深さ			Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数			k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧			E_a (kN/m)	
移動層の断面積	A (m²)	14190.30	二次元安全率			F_2	1.1966
地すべり抵抗力	ΣS (kN/m)	76365.94	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$				
地すべり力	ΣT (kN/m)	65943.58					

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	10.350	13.45	4.910	1600.89	36	29.990	22.530	20.00	20.460	12079.60
2	10.350	13.230	7.22	4.910	1957.85	37	22.530	18.800	10.00	20.460	4752.95
3	13.230	24.710	8.44	4.910	3682.46	38	18.800	12.520	6.89	42.340	2481.64
4	24.710	27.310	2.23	4.910	1334.05	39	12.520	10.000	2.76	42.340	714.78
5	27.310	27.870	12.96	4.910	8224.03	40	10.000	5.000	5.49	42.340	947.03
6	27.870	28.740	10.00	4.910	6510.15	41	5.000	0.000	5.49	42.340	315.68
7	28.740	33.260	12.60	4.910	8983.80						
8	33.260	39.530	8.43	4.910	7056.63						
9	39.530	40.900	8.98	4.910	8306.01						
10	40.900	42.610	13.36	4.910	12830.48						
11	42.610	41.170	16.88	4.910	16263.37						
12	41.170	43.090	12.53	4.910	12141.44						
13	43.090	42.140	11.03	4.910	10811.00						
14	42.140	38.580	6.60	4.910	6126.65						
15	38.580	37.740	19.59	4.910	17193.75						
16	37.740	37.050	20.00	4.910	17201.70						
17	37.050	36.000	20.00	4.910	16801.50						
18	36.000	30.550	20.00	13.790	15306.50						
19	30.550	24.970	20.00	13.790	12769.60						
20	24.970	22.040	8.02	13.790	4335.73						
21	22.040	18.890	7.96	13.790	3746.73						
22	18.890	21.440	21.88	13.790	10147.83						
23	21.440	21.650	2.14	13.790	1060.44						
24	21.650	27.800	23.63	13.790	13437.79						
25	27.800	37.380	16.37	13.790	12270.46						
26	37.380	42.590	13.94	13.790	12819.99						
27	42.590	45.590	8.12	13.790	8234.25						
28	45.590	49.130	5.97	13.790	6503.00						
29	49.130	48.640	1.97	13.790	2214.98						
30	48.640	46.560	5.59	20.460	6119.93						
31	46.560	46.120	1.18	20.460	1257.67						
32	46.120	43.810	6.18	20.460	6391.33						
33	43.810	41.260	6.83	20.460	6681.82						
34	41.260	37.870	9.09	20.460	8271.85						
35	37.870	29.990	21.13	20.460	16489.64						

備考：

NH54_priority 1
st.WS1

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流端 hc	
断面名	Cut_5	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
				末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	16.3977
地すべり抵抗力	Σ S (kN/m)	76365.94
地すべり力	Σ T (kN/m)	65943.58

設計済み負担推力			
分子側	アンカー	P_{sa} (kN/m)	
	杭	P_{sp} (kN/m)	
	その他	P_{sx} (kN/m)	
分母側	アンカー	P_{ma} (kN/m)	
	杭	P_{mp} (kN/m)	
	その他	P_{mx} (kN/m)	

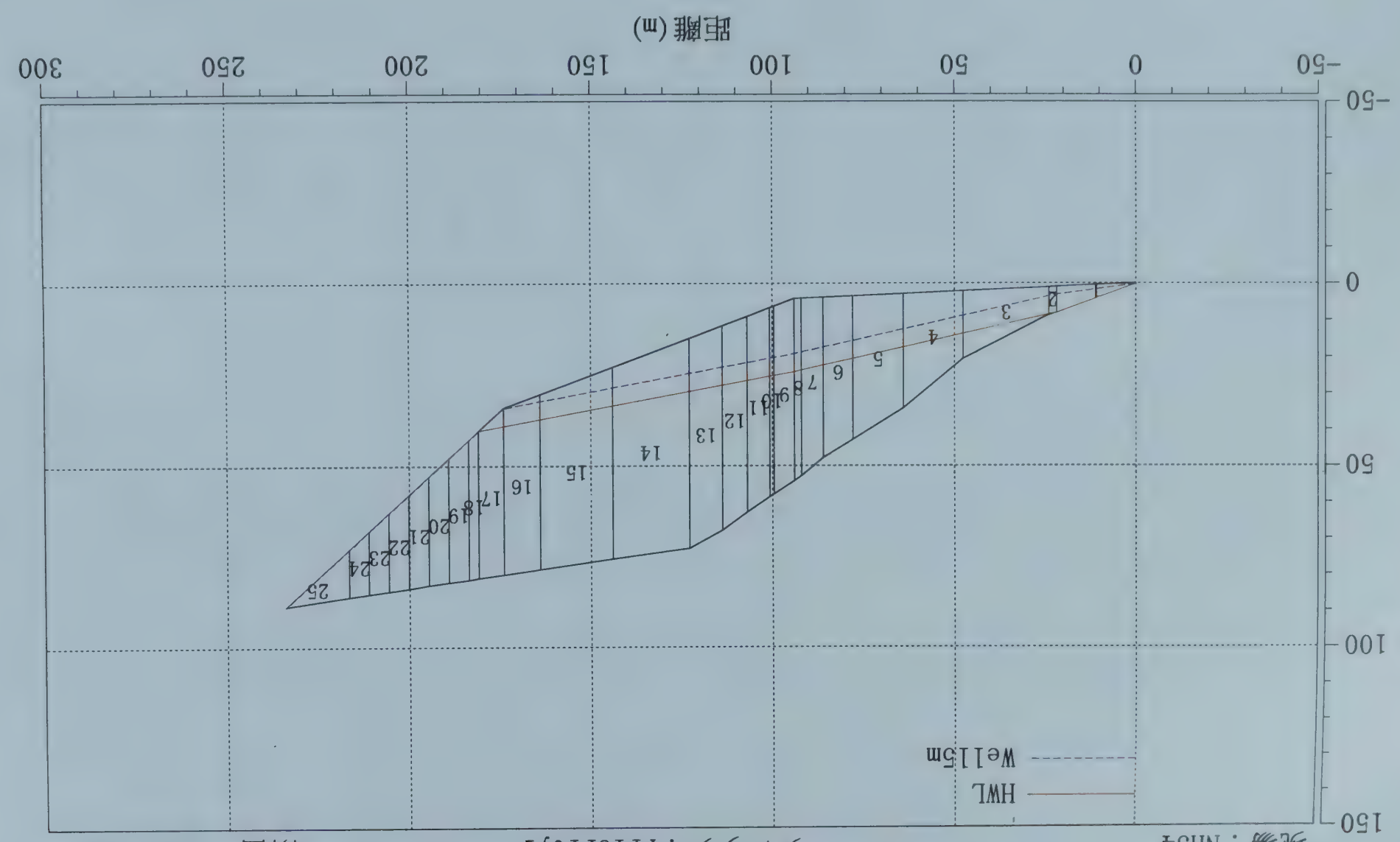
移動層の断面積	A (m^2)	14190.30
二次元安全率	F_2	1.1966

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	n_α	$S = s_l / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	10.350	13.45				682.6	13.50			270.23	1.0136	266.59	137.53
2	13.230	7.22				834.8	7.25			330.49	1.0136	326.04	168.19
3	24.710	8.44				1570.1	8.47			621.60	1.0136	613.24	316.35
4	24.620	2.23				539.4	2.24			233.85	1.0136	230.70	114.60
5	24.120	12.96				3097.3	13.01			1508.66	1.0136	1488.35	706.49
6	23.740	10.00				2346.7	10.04			1225.18	1.0136	1208.68	559.26
7	23.260	12.60				2903.7	12.65			1789.19	1.0136	1765.10	771.76
8	22.940	8.43				1909.7	8.46			1514.60	1.0136	1494.21	606.21
9	22.590	8.98				2004.8	9.01			1854.28	1.0136	1829.32	713.54
10	22.080	13.36				2926.3	13.41			2914.54	1.0136	2875.30	1102.22
11	21.440	16.88				3602.1	16.94			3725.87	1.0136	3675.71	1397.12
12	20.960	12.53				2605.0	12.58			2806.31	1.0136	2768.53	1043.02
13	20.530	11.03				2243.9	11.07			2521.05	1.0136	2487.11	928.73
14	20.280	6.60				1320.7	6.62			1414.26	1.0136	1395.22	526.32
15	19.530	19.59				3824.0	19.66			3934.35	1.0136	3881.38	1477.05
16	18.770	20.00				3755.9	20.07			3956.71	1.0136	3903.45	1477.73
17	18.000	20.00				3605.9	20.07			3883.10	1.0136	3830.82	1443.35
18	17.600	20.00				3491.2	20.59			3476.93	1.0001	3476.53	3756.80
19	17.200	20.00				3412.7	20.59			2753.47	1.0001	2753.16	3134.15
20	17.040	8.02				1346.5	8.26			879.66	1.0001	879.55	1064.16
21	16.880	7.96				1323.9	8.20			712.97	1.0001	712.89	919.59
22	16.440	21.88				3574.7	22.53			1934.28	1.0001	1934.06	2490.67
23	16.400	2.14				344.6	2.20			210.66	1.0001	210.63	260.27
24	15.930	23.63				3745.9	24.33			2852.05	1.0001	2851.72	3298.15
25	15.600	16.37				2530.8	16.86			2866.10	1.0001	2865.78	3011.64
26	15.320	13.94				2113.5	14.35			3150.64	1.0001	3150.28	3146.52
27	15.160	8.12				1213.6	8.36			2065.99	1.0001	2065.76	2021.00
28	15.040	5.97				884.0	6.15			1653.51	1.0001	1653.32	1596.09
29	15.000	1.97				290.2	2.03			566.42	1.0001	566.35	543.64
30	13.950	5.59				793.5	5.97			1567.42	0.9584	1635.53	2283.28
31	13.730	1.18				160.2	1.26			322.97	0.9584	337.00	469.22
32	12.570	6.18				797.0	6.60			1646.27	0.9584	1717.80	2384.53
33	11.290	6.83				799.1	7.29			1731.13	0.9584	1806.36	2492.92
34	9.590	9.09				930.6	9.70			2160.31	0.9584	2254.19	3086.14
35	5.630	21.13				1576.9	22.55			4388.41	0.9584	4579.10	6152.11
36	1.880	20.00				736.5	21.35			3337.97	0.9584	3483.01	4506.77
37	0.000	10.00				92.2	10.67			1371.54	0.9584	1431.13	1773.27
38						0.0	9.32			730.28	0.6688	1091.93	2261.29
39						0.0	3.73			210.34	0.6688	314.51	651.32
40						0.0	7.43			278.68	0.6688	416.69	862.94
41						0.0	7.43			92.89	0.6688	138.90	287.65

備考：

Appendix 5-3

Results of Stability Analysis
Main Survey Line L2 Upper



現場：NH54　　プロジェクト：Priority1　　断面：Isolated

縦断面図

縮尺：1/2000

A03-84

NH54_priority 1st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Isolated	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター	平均値			末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m³)	23.000	設計済み負担推力	
移動層の水中単位体積重量	γ' (kN/m³)		分子側	アンカー P_{sa} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m³)			杭 P_{sp} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)			その他 P_{sx} (kN/m)
すべり面材料の粘着力	c' (kN/m²)	0.0000	分母側	アンカー P_{ma} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	16.3977		杭 P_{mp} (kN/m)
弦～すべり面の最大深さ	d (m)	29.88		その他 P_{mx} (kN/m)
地すべり斜面弦長	L (m)	250.37	クラックの水位高	Z_w (m)
修正係数	f_0	1.0546	水圧の水平成分	Q_w (kN/m)
冠頭部クラックの傾斜角	θ_u (°)		陥没帯下流壁のすべり面深さ	Z_h (m)
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧係数	k_a
移動層の断面積	A (m²)	7613.02	主働土圧	E_a (kN/m)
地すべり抵抗力	ΣS (kN/m)	48592.18	二次元安全率	F_2 0.7627
地すべり力	ΣT (kN/m)	67191.26		

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	52.910	48.330	20.00	20.460	23285.20
2	6.950	7.590	2.14	2.550	357.83	16	48.330	46.000	10.00	20.460	10847.95
3	7.590	18.490	23.63	2.550	7087.11	17	46.000	40.670	6.89	42.340	6867.30
4	18.490	31.350	16.37	2.550	9382.63	18	40.670	38.630	2.76	42.340	2516.98
5	31.350	39.360	13.94	2.550	11335.52	19	38.630	34.380	5.49	42.340	4609.49
6	39.360	44.000	8.12	2.550	7784.16	20	34.380	30.100	5.49	42.340	4070.94
7	44.000	48.730	5.97	2.550	6366.38	21	30.100	26.000	5.49	42.340	3541.87
8	48.730	50.000	1.97	2.550	2236.73	22	26.000	21.790	5.49	42.340	3017.22
9	50.000	51.560	5.59	20.460	6528.78	23	21.790	17.560	5.49	42.340	2484.36
10	51.560	51.840	1.18	20.460	1403.14	24	17.560	13.300	5.49	42.340	1948.35
11	51.840	53.810	6.18	20.460	7508.55	25	13.300	0.000	17.49	42.340	2675.10
12	53.810	56.260	6.83	20.460	8645.45						
13	56.260	57.870	9.09	20.460	11930.58						
14	57.870	52.910	21.13	20.460	26918.99						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 ⊿x (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_σ	地すべり抵抗力 $S=s_l/n_\sigma$ (kN/m)	地すべり力 T (kN/m)
1	6.950	21.88				745.6	21.90			295.19	1.0152	290.78	77.88
2	7.340	2.14				149.9	2.14			61.17	1.0152	60.26	15.94
3	11.610	23.63				2195.7	23.65			1439.42	1.0152	1417.91	315.63
4	14.570	16.37				2101.4	16.39			2142.66	1.0152	2110.65	417.86
5	17.090	13.94				2164.0	13.95			2698.91	1.0152	2658.58	504.83
6	18.560	8.12				1419.4	8.13			1872.97	1.0152	1844.98	346.67
7	19.640	5.97				1118.2	5.98			1544.39	1.0152	1521.31	283.53
8	20.000	1.97				382.9	1.97			545.53	1.0152	537.38	99.61
9	18.950	5.59				1067.6	5.97			1607.07	1.0042	1600.39	2435.82
10	18.730	1.18				218.0	1.26			348.75	1.0042	347.30	523.50
11	17.570	6.18				1100.0	6.60			1885.86	1.0042	1878.02	2801.36
12	16.290	6.83				1134.0	7.29			2210.42	1.0042	2201.23	3225.52
13	14.590	9.09				1376.4	9.70			3105.82	1.0042	3092.90	4451.17
14	10.620	21.13				2611.9	22.55			7152.89	1.0042	7123.16	10043.18
15	6.870	20.00				1715.2	21.35			6347.46	1.0042	6321.07	8687.46
16	5.000	10.00				582.0	10.67			3020.98	1.0042	3008.42	4047.25
17	0.000	6.89				168.9	9.32			1971.15	0.7384	2669.32	6257.53
18						0.0	3.73			740.68	0.7384	1003.02	2293.49
19						0.0	7.43			1356.44	0.7384	1836.89	4200.20
20						0.0	7.43			1197.97	0.7384	1622.28	3709.47
21						0.0	7.43			1042.28	0.7384	1411.45	3227.38
22						0.0	7.43			887.89	0.7384	1202.37	2749.31
23						0.0	7.43			731.08	0.7384	990.02	2263.77
24						0.0	7.43			573.34	0.7384	776.42	1775.35
25						0.0	23.66			787.21	0.7384	1066.03	2437.57

備考

NH54_priority 1
st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Isolated	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000		設計済み負担推力 分子側	アンカー P_{sa} (kN/m)		
移動層の水中単位体積重量	γ' (kN/m³)				杭 P_{sp} (kN/m)		
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他 P_{sx} (kN/m)		
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)		
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭 P_{mp} (kN/m)		
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他 P_{mx} (kN/m)		
弦～すべり面の最大深さ	d (m)	29.88		クラックの水位高	Z_w (m)		
地すべり斜面弦長	L (m)	250.37		水圧の水平成分	Q_w (kN/m)		
修正係数	f_0	1.0546		陥没帯下流壁のすべり面深さ	Z_h (m)		
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数	k_a		
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧	E_a (kN/m)		
移動層の断面積	A (m²)	7613.02		二次元安全率	F_2	0.8062	
地すべり抵抗力	ΣS (kN/m)	51361.89					
地すべり力	ΣT (kN/m)	67191.26					

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

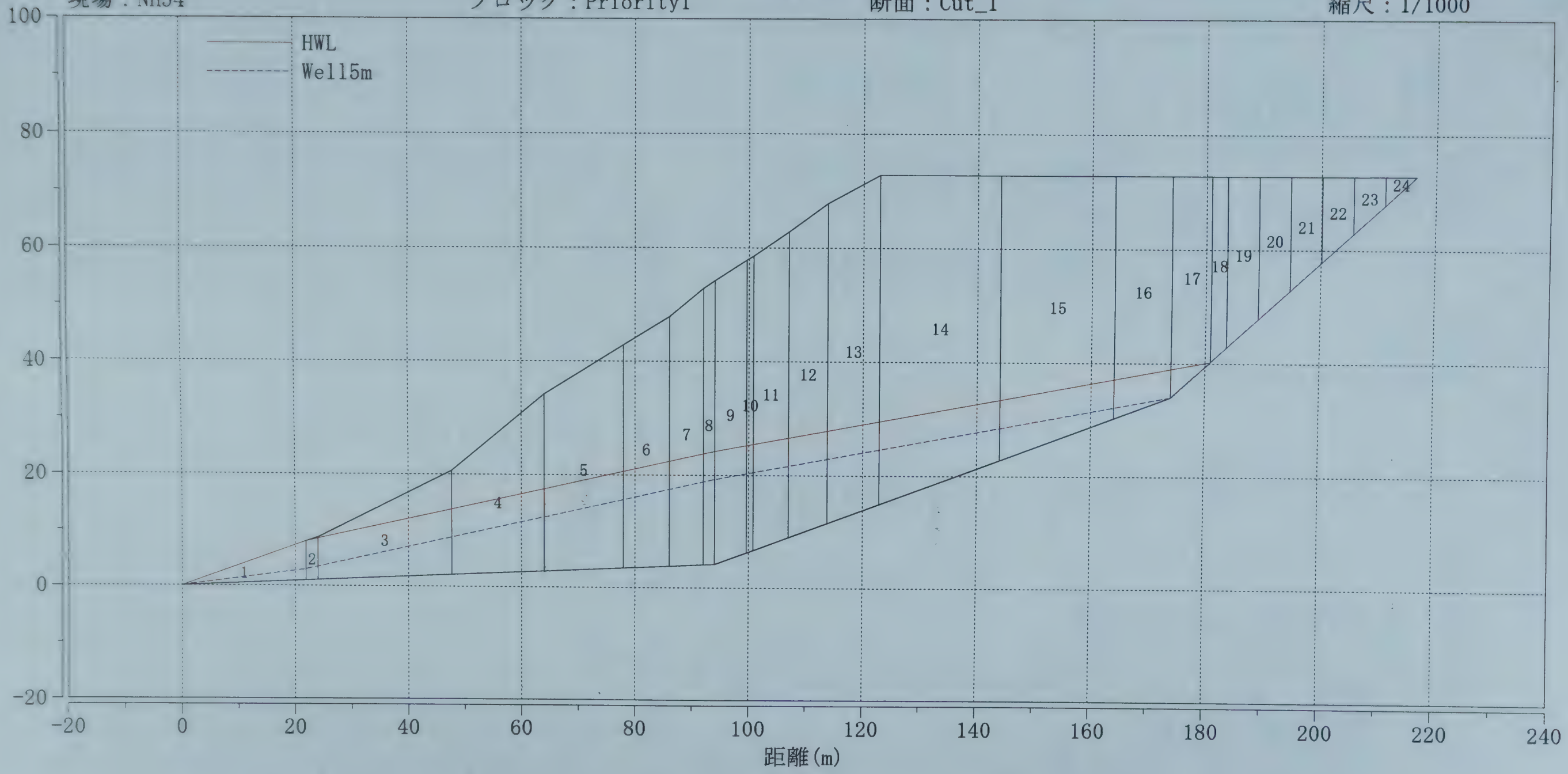
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	52.910	48.330	20.00	20.460	23285.20
2	6.950	7.590	2.14	2.550	357.83	16	48.330	46.000	10.00	20.460	10847.95
3	7.590	18.490	23.63	2.550	7087.11	17	46.000	40.670	6.89	42.340	6867.30
4	18.490	31.350	16.37	2.550	9382.63	18	40.670	38.630	2.76	42.340	2516.98
5	31.350	39.360	13.94	2.550	11335.52	19	38.630	34.380	5.49	42.340	4609.49
6	39.360	44.000	8.12	2.550	7784.16	20	34.380	30.100	5.49	42.340	4070.94
7	44.000	48.730	5.97	2.550	6366.38	21	30.100	26.000	5.49	42.340	3541.87
8	48.730	50.000	1.97	2.550	2236.73	22	26.000	21.790	5.49	42.340	3017.22
9	50.000	51.560	5.59	20.460	6528.78	23	21.790	17.560	5.49	42.340	2484.36
10	51.560	51.840	1.18	20.460	1403.14	24	17.560	13.300	5.49	42.340	1948.35
11	51.840	53.810	6.18	20.460	7508.55	25	13.300	0.000	17.49	42.340	2675.10
12	53.810	56.260	6.83	20.460	8645.45						
13	56.260	57.870	9.09	20.460	11930.58						
14	57.870	52.910	21.13	20.460	26918.99						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0142	446.69	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0142	90.76	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0142	1755.38	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0142	2345.46	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0142	2859.33	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0142	1962.18	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0142	1607.63	283.53
8	15.000	1.97				286.3	1.97			573.95	1.0142	565.89	99.61
9	13.950	5.59				793.5	5.97			1687.73	0.9974	1692.20	2435.82
10	13.730	1.18				160.2	1.26			365.78	0.9974	366.74	523.50
11	12.570	6.18				797.0	6.60			1975.04	0.9974	1980.26	2801.36
12	11.290	6.83				799.1	7.29			2308.97	0.9974	2315.08	3225.52
13	9.590	9.09				930.6	9.70			3236.98	0.9974	3245.54	4451.17
14	5.630	21.13				1576.9	22.55			7457.48	0.9974	7477.21	10043.18
15	1.880	20.00				736.5	21.35			6635.47	0.9974	6653.03	8687.46
16	0.000	10.00				92.2	10.67			3165.13	0.9974	3173.50	4047.25
17						0.0	9.32			2020.86	0.7281	2775.57	6257.53
18						0.0	3.73			740.68	0.7281	1017.29	2293.49
19						0.0	7.43			1356.44	0.7281	1863.02	4200.20
20						0.0	7.43			1197.97	0.7281	1645.36	3709.47
21						0.0	7.43			1042.28	0.7281	1431.52	3227.38
22						0.0	7.43			887.89	0.7281	1219.48	2749.31
23						0.0	7.43			731.08	0.7281	1004.11	2263.77
24						0.0	7.43			573.34	0.7281	787.47	1775.35
25						0.0	23.66			787.21	0.7281	1081.20	2437.57

備考

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_1　　縮尺：1/1000



A03-87

NH54_priority 1 st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_1	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	26.00		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	228.53		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0531		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	6876.41		二次元安全率		F_2	0.8860
地すべり抵抗力	ΣS (kN/m)	45447.51					
地すべり力	ΣT (kN/m)	54019.12					

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

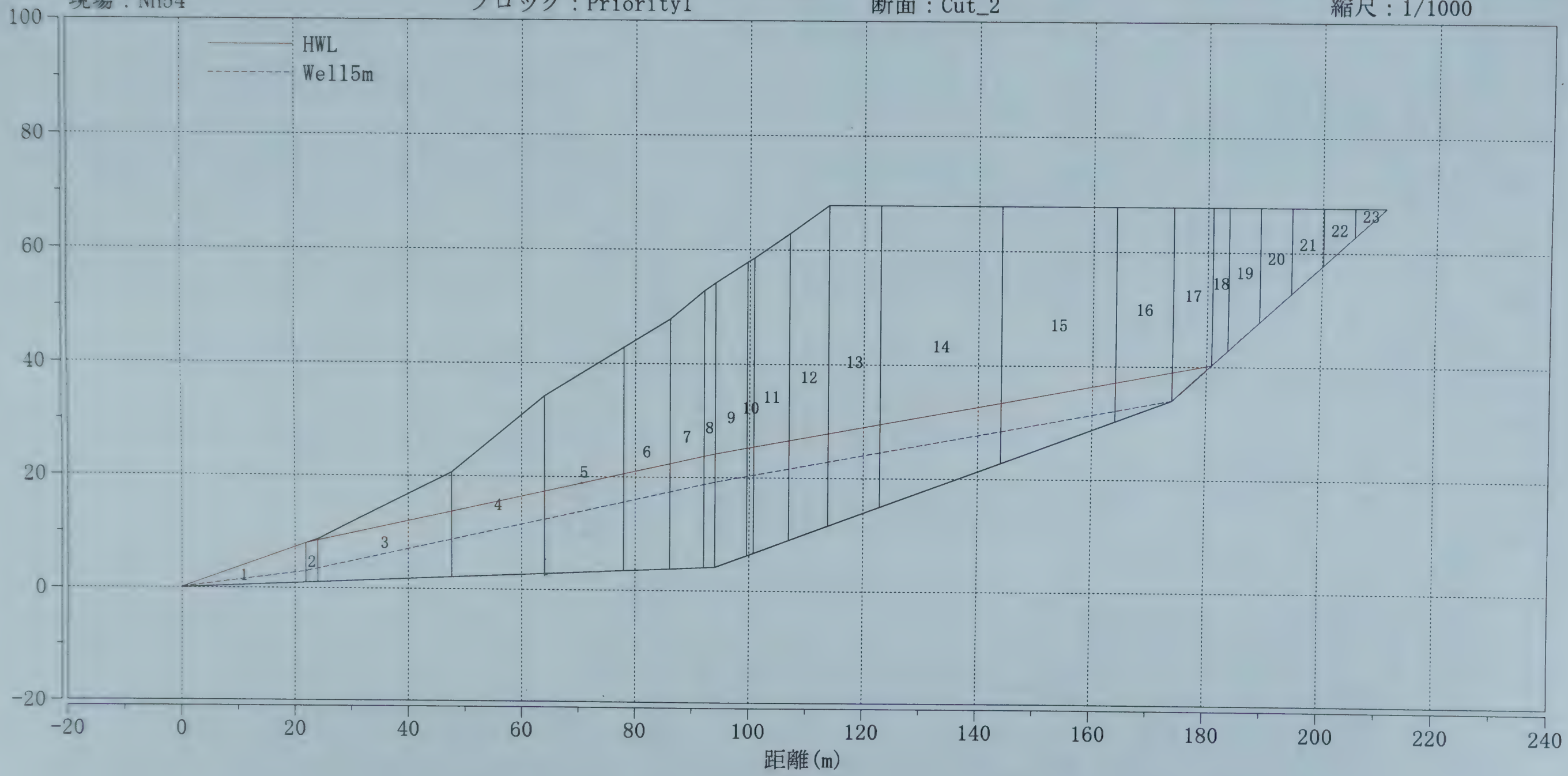
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	49.990	42.530	20.00	20.460	21279.60
2	6.950	7.590	2.14	2.550	357.83	16	42.530	38.800	10.00	20.460	9352.95
3	7.590	18.490	23.63	2.550	7087.11	17	38.800	32.520	6.89	42.340	5651.04
4	18.490	31.350	16.37	2.550	9382.63	18	32.520	30.000	2.76	42.340	1984.38
5	31.350	39.360	13.94	2.550	11335.52	19	30.000	25.000	5.49	42.340	3472.43
6	39.360	44.000	8.12	2.550	7784.16	20	25.000	20.000	5.49	42.340	2841.08
7	44.000	48.730	5.97	2.550	6366.38	21	20.000	15.000	5.49	42.340	2209.73
8	48.730	50.000	1.97	2.550	2236.73	22	15.000	10.000	5.49	42.340	1578.38
9	50.000	51.560	5.59	20.460	6528.78	23	10.000	5.000	5.49	42.340	947.03
10	51.560	51.840	1.18	20.460	1403.14	24	5.000	0.000	5.49	42.340	315.68
11	51.840	53.810	6.18	20.460	7508.55						
12	53.810	56.260	6.83	20.460	8645.45						
13	56.260	57.870	9.09	20.460	11930.58						
14	57.870	49.990	21.13	20.460	26209.44						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0128	447.33	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0128	90.89	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0128	1757.91	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0128	2348.84	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0128	2863.45	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0128	1965.02	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0128	1609.95	283.53
8	15.000	1.97				286.3	1.97			573.95	1.0128	566.71	99.61
9	13.950	5.59				793.5	5.97			1687.73	0.9866	1710.68	2435.82
10	13.730	1.18				160.2	1.26			365.78	0.9866	370.75	523.50
11	12.570	6.18				797.0	6.60			1975.04	0.9866	2001.88	2801.36
12	11.290	6.83				799.1	7.29			2308.97	0.9866	2340.36	3225.52
13	9.590	9.09				930.6	9.70			3236.98	0.9866	3280.98	4451.17
14	5.630	21.13				1576.9	22.55			7248.68	0.9866	7347.22	9778.46
15	1.880	20.00				736.5	21.35			6045.27	0.9866	6127.46	7939.19
16	0.000	10.00				92.2	10.67			2725.19	0.9866	2762.24	3489.48
17						0.0	9.32			1662.95	0.7117	2336.54	5149.27
18						0.0	3.73			583.95	0.7117	820.49	1808.19
19						0.0	7.43			1021.84	0.7117	1435.75	3164.10
20						0.0	7.43			836.05	0.7117	1174.70	2588.81
21						0.0	7.43			650.26	0.7117	913.66	2013.52
22						0.0	7.43			464.47	0.7117	652.61	1438.23
23						0.0	7.43			278.68	0.7117	391.57	862.94
24						0.0	7.43			92.89	0.7117	130.52	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_2　　縮尺：1/1000



NH54_priority 1
st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_2	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000		設計済み負担推力 分子側	アンカー	P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)				杭	P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)				その他	P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー	P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000			杭	P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977			その他	P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	24.78		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	221.75		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0525		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	6398.81		二次元安全率		F_2	0.9279
地すべり抵抗力	ΣS (kN/m)	41834.18					
地すべり力	ΣT (kN/m)	47455.06					

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

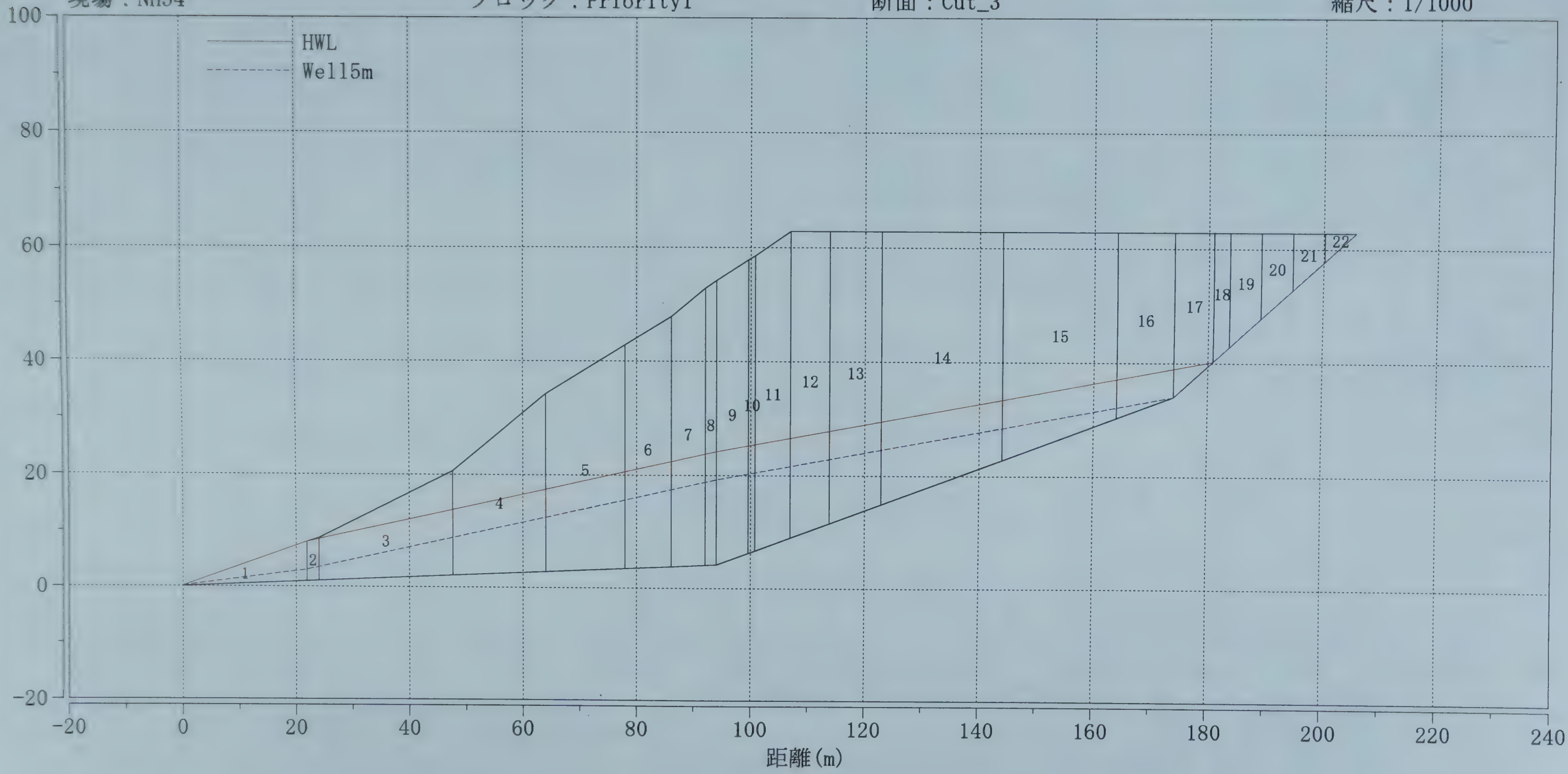
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	44.990	37.530	20.00	20.460	18979.60
2	6.950	7.590	2.14	2.550	357.83	16	37.530	33.800	10.00	20.460	8202.95
3	7.590	18.490	23.63	2.550	7087.11	17	33.800	27.520	6.89	42.340	4858.69
4	18.490	31.350	16.37	2.550	9382.63	18	27.520	25.000	2.76	42.340	1666.98
5	31.350	39.360	13.94	2.550	11335.52	19	25.000	20.000	5.49	42.340	2841.08
6	39.360	44.000	8.12	2.550	7784.16	20	20.000	15.000	5.49	42.340	2209.73
7	44.000	48.730	5.97	2.550	6366.38	21	15.000	10.000	5.49	42.340	1578.38
8	48.730	50.000	1.97	2.550	2236.73	22	10.000	5.000	5.49	42.340	947.03
9	50.000	51.560	5.59	20.460	6528.78	23	5.000	0.000	5.49	42.340	315.68
10	51.560	51.840	1.18	20.460	1403.14						
11	51.840	53.810	6.18	20.460	7508.55						
12	53.810	56.260	6.83	20.460	8645.45						
13	56.260	52.870	9.09	20.460	11407.90						
14	52.870	44.990	21.13	20.460	23779.49						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0121	447.62	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0121	90.95	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0121	1759.07	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0121	2350.39	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0121	2865.34	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0121	1966.31	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0121	1611.01	283.53
8	15.000	1.97				286.3	1.97			573.95	1.0121	567.08	99.61
9	13.950	5.59				793.5	5.97			1687.73	0.9817	1719.23	2435.82
10	13.730	1.18				160.2	1.26			365.78	0.9817	372.60	523.50
11	12.570	6.18				797.0	6.60			1975.04	0.9817	2011.89	2801.36
12	11.290	6.83				799.1	7.29			2308.97	0.9817	2352.07	3225.52
13	9.590	9.09				930.6	9.70			3083.17	0.9817	3140.71	4256.17
14	5.630	21.13				1576.9	22.55			6533.61	0.9817	6655.54	8871.87
15	1.880	20.00				736.5	21.35			5368.45	0.9817	5468.64	7081.08
16	0.000	10.00				92.2	10.67			2386.77	0.9817	2431.32	3060.43
17						0.0	9.32			1429.78	0.7042	2030.21	4427.27
18						0.0	3.73			490.55	0.7042	696.55	1518.97
19						0.0	7.43			836.05	0.7042	1187.15	2588.81
20						0.0	7.43			650.26	0.7042	923.34	2013.52
21						0.0	7.43			464.47	0.7042	659.53	1438.23
22						0.0	7.43			278.68	0.7042	395.72	862.94
23						0.0	7.43			92.89	0.7042	131.91	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_3　　縮尺：1/1000



A03-91

NH54_priority 1 st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1
断面名	Cut_3	間隙水圧名	Well5m
計算条件			

安定解析式	簡易Janbu法	陥没帯下流壁 No.	
地下水条件	被圧地下水		
水平外力 Q(kN/m)	0.00	水平震度 k_h	
湛水条件	なし	基準水面の標高BL(m)	
末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	16.3977
弦～すべり面の最大深さ	d (m)	23.47
地すべり斜面弦長	L (m)	215.02
修正係数	f_0	1.0518
冠頭部クラックの傾斜角	θ_u (°)	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)	
移動層の断面積	A (m^2)	5908.86
地すべり抵抗力	Σ S (kN/m)	38161.80
地すべり力	Σ T (kN/m)	41124.77

設計済み負担推力				
分子側	アンカー	P_{sa}	(kN/m)	
	杭	P_{sp}	(kN/m)	
	その他	P_{sx}	(kN/m)	
分母側	アンカー	P_{ma}	(kN/m)	
	杭	P_{mp}	(kN/m)	
	その他	P_{mx}	(kN/m)	

クラックの水位高	Z_w	(m)	
水圧の水平成分	Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ	Z_h	(m)	
主働土圧係数	k_a		
主働土圧	E_a	(kN/m)	
二次元安全率	F_2		0.9760

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

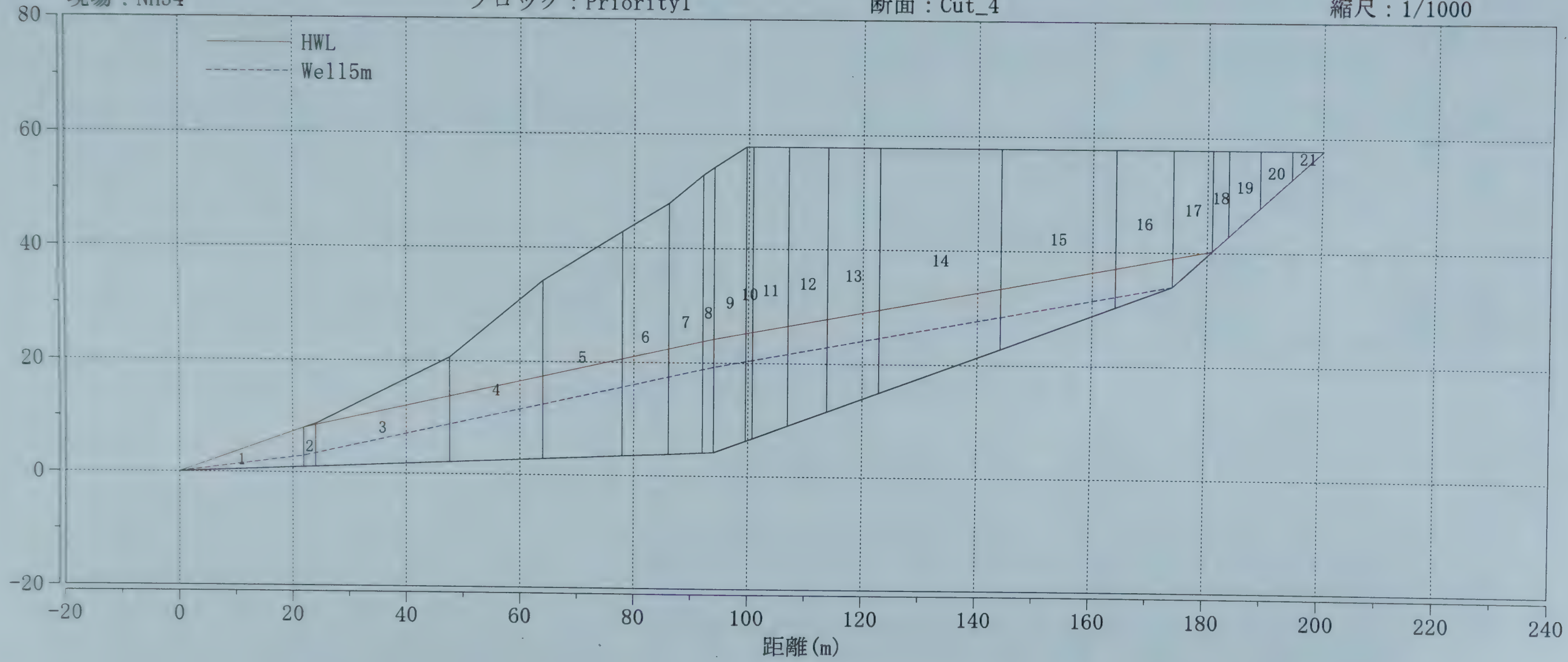
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	39.990	32.530	20.00	20.460	16679.60
2	6.950	7.590	2.14	2.550	357.83	16	32.530	28.800	10.00	20.460	7052.95
3	7.590	18.490	23.63	2.550	7087.11	17	28.800	22.520	6.89	42.340	4066.34
4	18.490	31.350	16.37	2.550	9382.63	18	22.520	20.000	2.76	42.340	1349.58
5	31.350	39.360	13.94	2.550	11335.52	19	20.000	15.000	5.49	42.340	2209.73
6	39.360	44.000	8.12	2.550	7784.16	20	15.000	10.000	5.49	42.340	1578.38
7	44.000	48.730	5.97	2.550	6366.38	21	10.000	5.000	5.49	42.340	947.03
8	48.730	50.000	1.97	2.550	2236.73	22	5.000	0.000	5.49	42.340	315.68
9	50.000	51.560	5.59	20.460	6528.78						
10	51.560	51.840	1.18	20.460	1403.14						
11	51.840	53.810	6.18	20.460	7508.55						
12	53.810	51.260	6.83	20.460	8252.72						
13	51.260	47.870	9.09	20.460	10362.55						
14	47.870	39.990	21.13	20.460	21349.54						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 S=s_1 / n_α (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0114	447.93	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0114	91.01	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0114	1760.28	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0114	2352.01	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0114	2867.31	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0114	1967.66	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0114	1612.12	283.53
8	15.000	1.97				286.3	1.97			573.95	1.0114	567.47	99.61
9	13.950	5.59				793.5	5.97			1687.73	0.9766	1728.25	2435.82
10	13.730	1.18				160.2	1.26			365.78	0.9766	374.56	523.50
11	12.570	6.18				797.0	6.60			1975.04	0.9766	2022.45	2801.36
12	11.290	6.83				799.1	7.29			2193.41	0.9766	2246.07	3079.00
13	9.590	9.09				930.6	9.70			2775.55	0.9766	2842.19	3866.16
14	5.630	21.13				1576.9	22.55			5818.54	0.9766	5958.24	7965.28
15	1.880	20.00				736.5	21.35			4691.62	0.9766	4804.26	6222.98
16	0.000	10.00				92.2	10.67			2048.36	0.9766	2097.54	2631.38
17						0.0	9.32			1196.61	0.6965	1718.13	3705.28
18						0.0	3.73			397.15	0.6965	570.23	1229.75
19						0.0	7.43			650.26	0.6965	933.67	2013.52
20						0.0	7.43			464.47	0.6965	666.90	1438.23
21						0.0	7.43			278.68	0.6965	400.14	862.94
22						0.0	7.43			92.89	0.6965	133.38	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_4　　縮尺：1/1000



A03-93

NH54_priority 1st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁	No.
断面名	Cut_4	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
				末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	23.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	16.3977
弦～すべり面の最大深さ	d (m)	22.08
地すべり斜面弦長	L (m)	208.33
修正係数	f_0	1.0509
冠頭部クラックの傾斜角	θ_u (°)	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)	
移動層の断面積	A (m^2)	5411.18
地すべり抵抗力	Σ S (kN/m)	34464.65
地すべり力	Σ T (kN/m)	35067.93

設計済み負担推力			
分子側	アンカー	P_{sa}	(kN/m)
	杭	P_{sp}	(kN/m)
	その他	P_{sx}	(kN/m)
分母側	アンカー	P_{ma}	(kN/m)
	杭	P_{mp}	(kN/m)
	その他	P_{mx}	(kN/m)

クラックの水位高	Z_w	(m)	
水圧の水平成分	Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ	Z_h	(m)	
主働土圧係数	k_a		
主働土圧	E_a	(kN/m)	
二次元安全率	F_2		1.0328

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

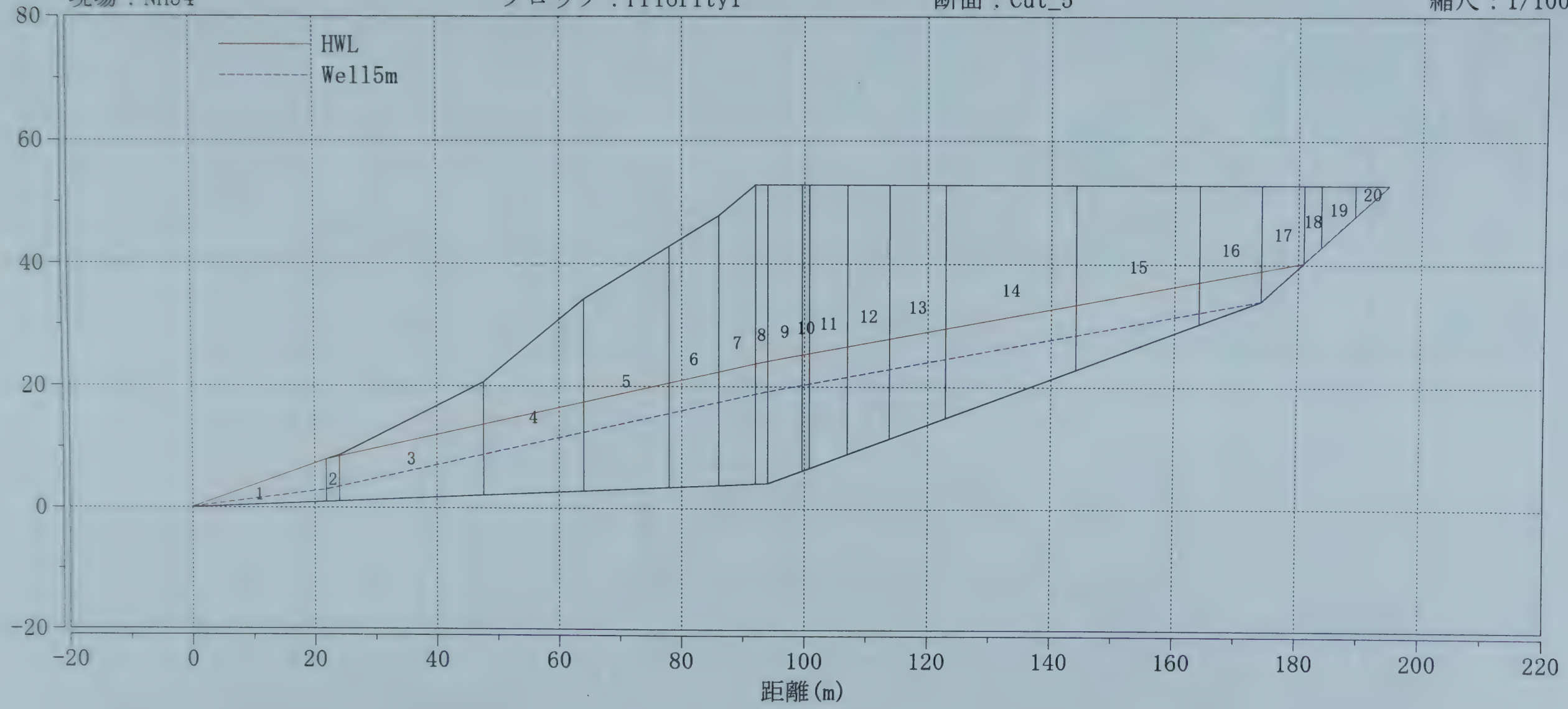
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	34.990	27.530	20.00	20.460	14379.60
2	6.950	7.590	2.14	2.550	357.83	16	27.530	23.800	10.00	20.460	5902.95
3	7.590	18.490	23.63	2.550	7087.11	17	23.800	17.520	6.89	42.340	3273.99
4	18.490	31.350	16.37	2.550	9382.63	18	17.520	15.000	2.76	42.340	1032.18
5	31.350	39.360	13.94	2.550	11335.52	19	15.000	10.000	5.49	42.340	1578.38
6	39.360	44.000	8.12	2.550	7784.16	20	10.000	5.000	5.49	42.340	947.03
7	44.000	48.730	5.97	2.550	6366.38	21	5.000	0.000	5.49	42.340	315.68
8	48.730	50.000	1.97	2.550	2236.73						
9	50.000	51.560	5.59	20.460	6528.78						
10	51.560	51.120	1.18	20.460	1393.37						
11	51.120	48.810	6.18	20.460	7102.03						
12	48.810	46.260	6.83	20.460	7467.27						
13	46.260	42.870	9.09	20.460	9317.20						
14	42.870	34.990	21.13	20.460	18919.59						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0107	448.26	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0107	91.08	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0107	1761.56	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0107	2353.72	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0107	2869.40	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0107	1969.10	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0107	1613.29	283.53
8	15.000	1.97				286.3	1.97			573.95	1.0107	567.89	99.61
9	13.950	5.59				793.5	5.97			1687.73	0.9711	1737.91	2435.82
10	13.730	1.18				160.2	1.26			362.90	0.9711	373.69	519.85
11	12.570	6.18				797.0	6.60			1855.41	0.9711	1910.57	2649.69
12	11.290	6.83				799.1	7.29			1962.27	0.9711	2020.61	2785.96
13	9.590	9.09				930.6	9.70			2467.93	0.9711	2541.31	3476.15
14	5.630	21.13				1576.9	22.55			5103.47	0.9711	5255.21	7058.69
15	1.880	20.00				736.5	21.35			4014.79	0.9711	4134.16	5364.87
16	0.000	10.00				92.2	10.67			1709.95	0.9711	1760.79	2202.33
17						0.0	9.32			963.44	0.6882	1399.93	2983.28
18						0.0	3.73			303.74	0.6882	441.35	940.53
19						0.0	7.43			464.47	0.6882	674.90	1438.23
20						0.0	7.43			278.68	0.6882	404.94	862.94
21						0.0	7.43			92.89	0.6882	134.98	287.65

備考：

縦断面図

現場：NH54　　ブロック：Priority1　　断面：Cut_5　　縮尺：1/1000



A03-95

NH54_priority 1 st_Upper_isolat

二 次 元 安 定 解 析 表

現場名	NH54	ブロック名	Priority1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut_5	間隙水圧名	Well5m	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	23.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	16.3977				その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	20.59		クラックの水位高	Z_w (m)		
地すべり斜面弦長	L (m)	201.69		水圧の水平成分	Q_w (kN/m)		
修正係数	f_0	1.0497		陥没帯下流壁のすべり面深さ	Z_h (m)		
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数	k_a		
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧	E_a (kN/m)		
移動層の断面積	A (m²)	4903.14		二次元安全率	F_2		1.1019
地すべり抵抗力	ΣS (kN/m)	30726.61		$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	ΣT (kN/m)	29272.00					

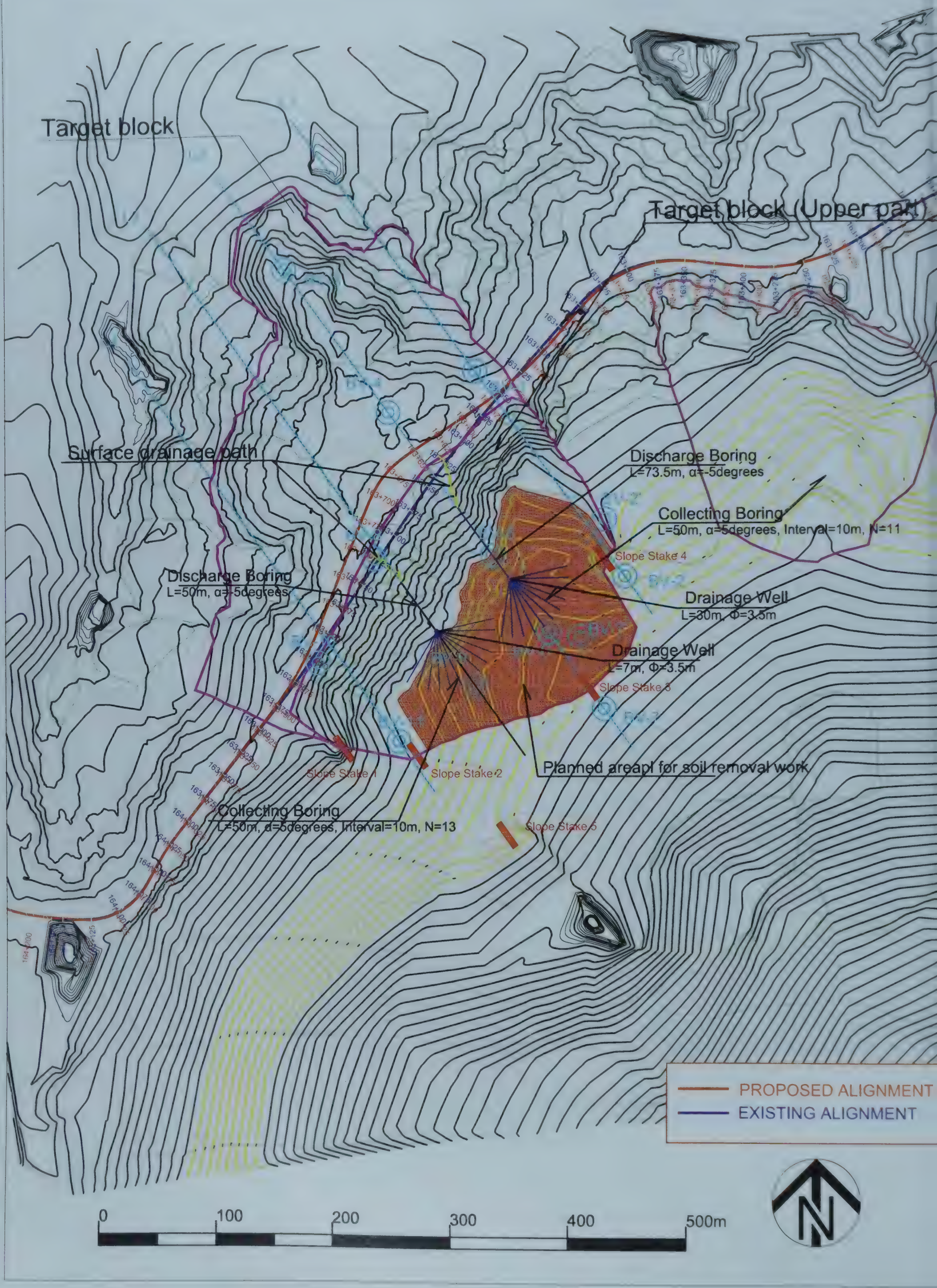
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	6.950	21.88	2.550	1748.76	15	29.990	22.530	20.00	20.460	12079.60
2	6.950	7.590	2.14	2.550	357.83	16	22.530	18.800	10.00	20.460	4752.95
3	7.590	18.490	23.63	2.550	7087.11	17	18.800	12.520	6.89	42.340	2481.64
4	18.490	31.350	16.37	2.550	9382.63	18	12.520	10.000	2.76	42.340	714.78
5	31.350	39.360	13.94	2.550	11335.52	19	10.000	5.000	5.49	42.340	947.03
6	39.360	44.000	8.12	2.550	7784.16	20	5.000	0.000	5.49	42.340	315.68
7	44.000	48.730	5.97	2.550	6366.38						
8	48.730	48.640	1.97	2.550	2205.92						
9	48.640	46.560	5.59	20.460	6119.93						
10	46.560	46.120	1.18	20.460	1257.67						
11	46.120	43.810	6.18	20.460	6391.33						
12	43.810	41.260	6.83	20.460	6681.82						
13	41.260	37.870	9.09	20.460	8271.85						
14	37.870	29.990	21.13	20.460	16489.64						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.950	21.88				209.2	21.90			453.05	1.0099	448.61	77.88
2	2.340	2.14				45.0	2.14			92.05	1.0099	91.15	15.94
3	6.610	23.63				1037.0	23.65			1780.38	1.0099	1762.94	315.63
4	9.570	16.37				1298.7	16.39			2378.87	1.0099	2355.57	417.86
5	12.090	13.94				1480.5	13.95			2900.06	1.0099	2871.65	504.83
6	13.560	8.12				1021.3	8.13			1990.14	1.0099	1970.64	346.67
7	14.640	5.97				825.5	5.98			1630.53	1.0099	1614.56	283.53
8	15.000	1.97				286.3	1.97			564.89	1.0099	559.36	98.24
9	13.950	5.59				793.5	5.97			1567.42	0.9653	1623.80	2283.28
10	13.730	1.18				160.2	1.26			322.97	0.9653	334.59	469.22
11	12.570	6.18				797.0	6.60			1646.27	0.9653	1705.49	2384.53
12	11.290	6.83				799.1	7.29			1731.13	0.9653	1793.40	2492.92
13	9.590	9.09				930.6	9.70			2160.31	0.9653	2238.02	3086.14
14	5.630	21.13				1576.9	22.55			4388.41	0.9653	4546.26	6152.11
15	1.880	20.00				736.5	21.35			3337.97	0.9653	3458.04	4506.77
16	0.000	10.00				92.2	10.67			1371.54	0.9653	1420.87	1773.27
17						0.0	9.32			730.28	0.6793	1075.02	2261.29
18						0.0	3.73			210.34	0.6793	309.64	651.32
19						0.0	7.43			278.68	0.6793	410.24	862.94
20						0.0	7.43			92.89	0.6793	136.75	287.65

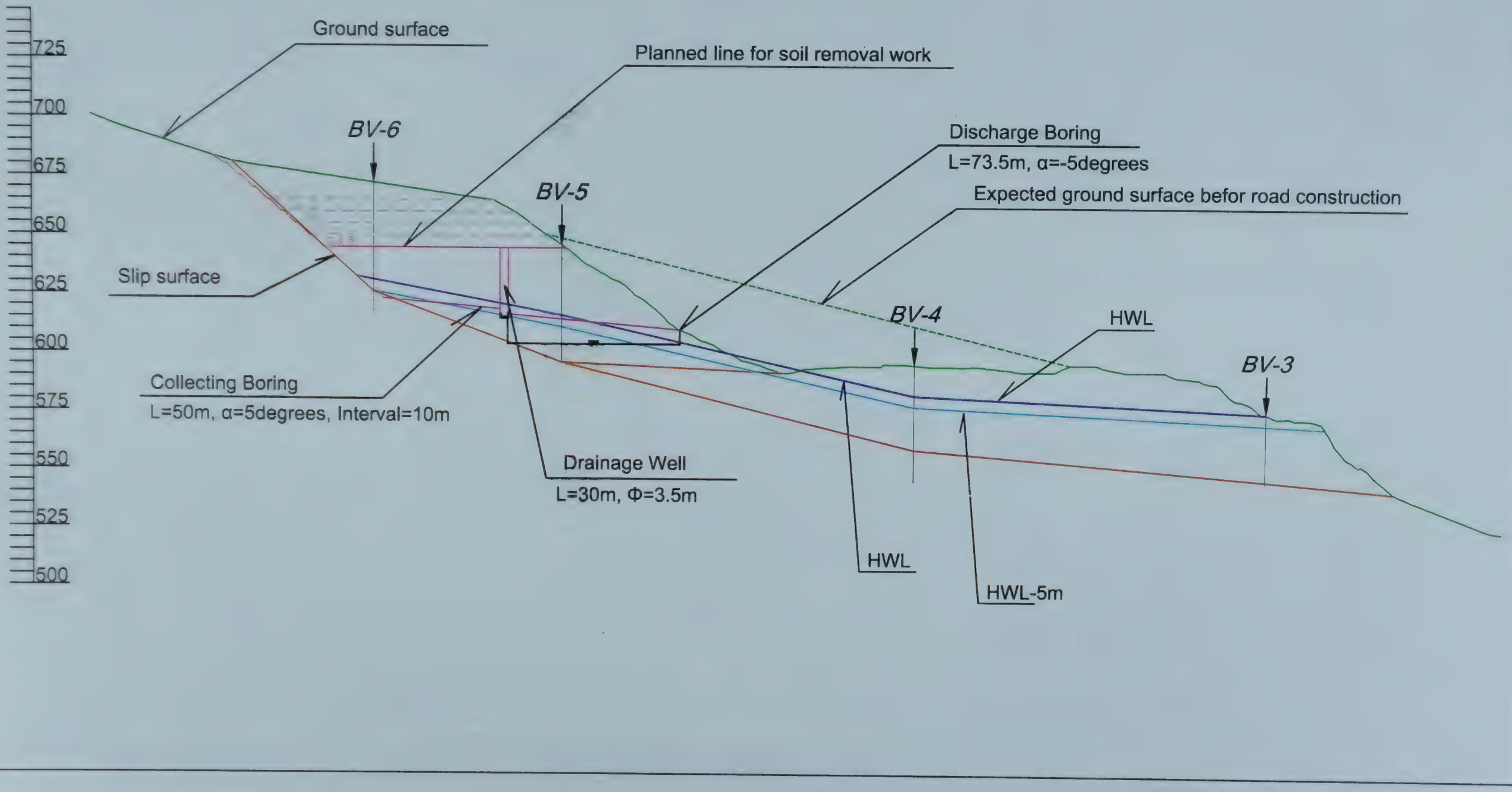
備考：

Appendix 6

Plan and Profile of Recommendation


Target block
Target block (Upper part)
Surface drainage path
Discharge Boring
L=73.5m, α=-5degrees
Collecting Boring
L=50m, α=5degrees, Interval=10m, N=11
Slope Stake 4
Discharge Boring
L=50m, α=-5degrees
Drainage Well
L=30m, Φ=3.5m
Drainage Well
L=7m, Φ=3.5m
Slope Stake 3
Slope Stake 1
Slope Stake 2
Planned areapl for soil removal work
Collecting Boring
L=50m, α=5degrees, Interval=10m, N=13
Slope Stake 5
PROPOSED ALIGNMENT
EXISTING ALIGNMENT
0
100
200
300
400
500m
N

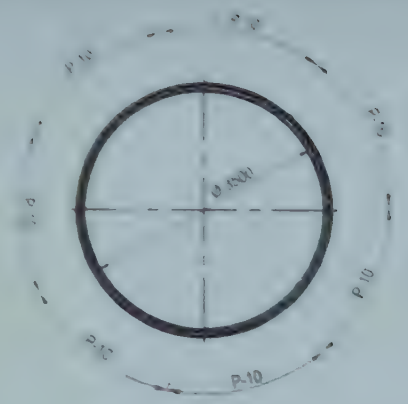
CROSS SECTION OF SURVEY LINE L2



A03-99

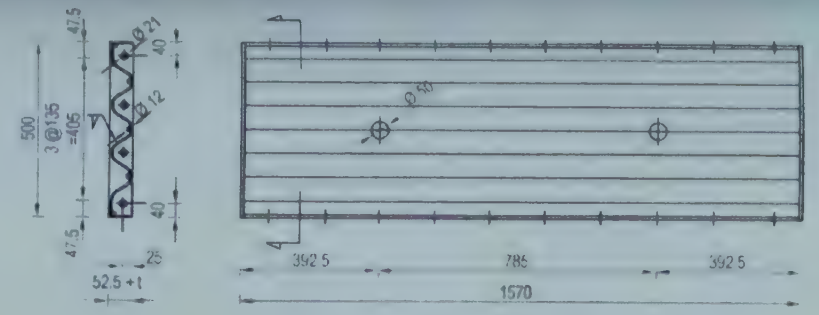
Appendix 7

Image Drawing of Drainage Well

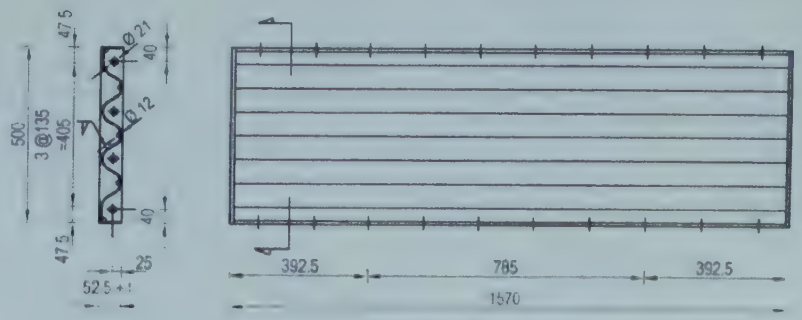


LAYOUT PLAN OF LINER PLATE
SCALE 1:100

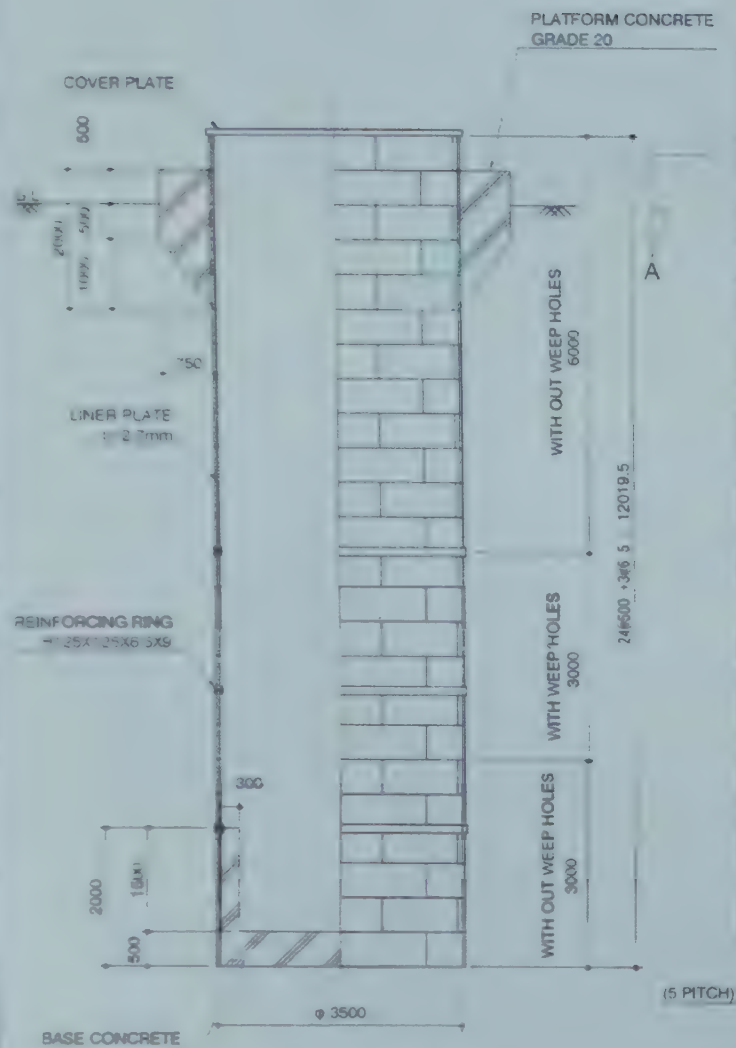
SECTION A-A PLAN OF LADDER
SCALE 1:100



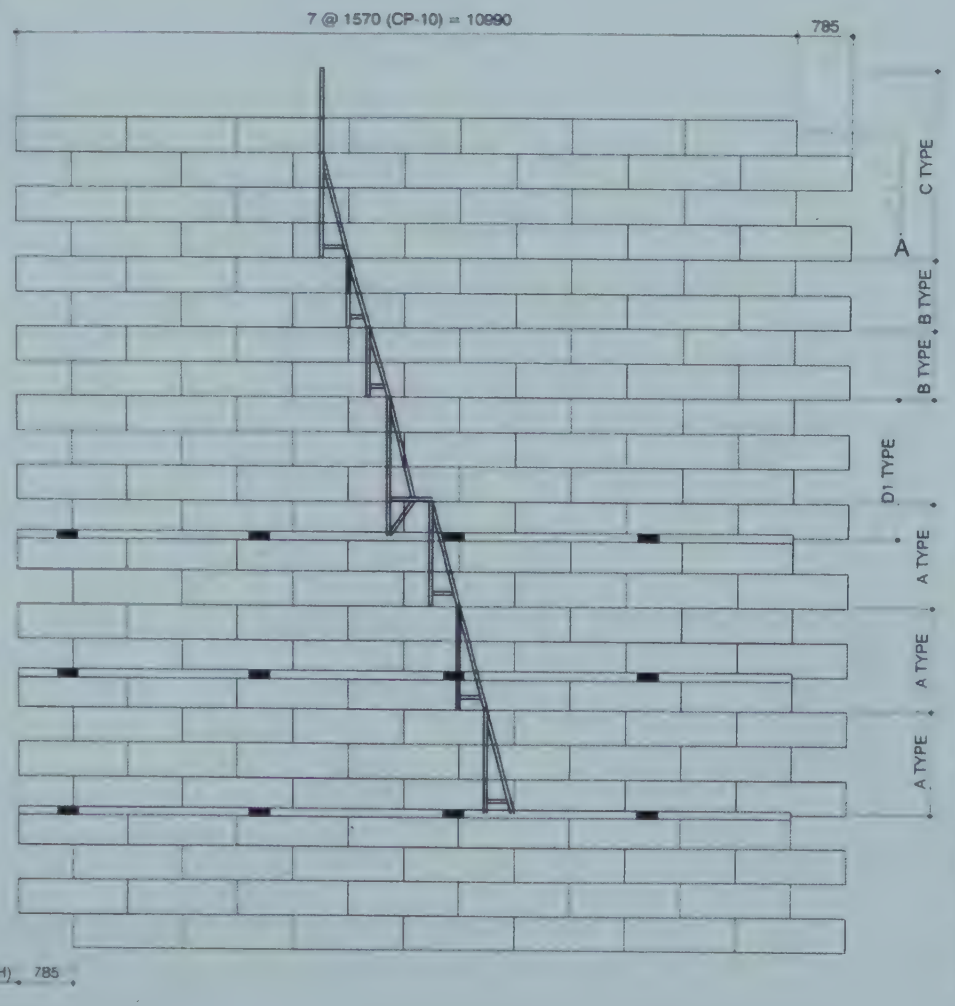
LINER PLATE WITH WEEP HOLES
SCALE 1:20



LINER PLATE WITHOUT WEEP HOLES
SCALE 1:20



SECTIONAL PLAN
SCALE 1:100



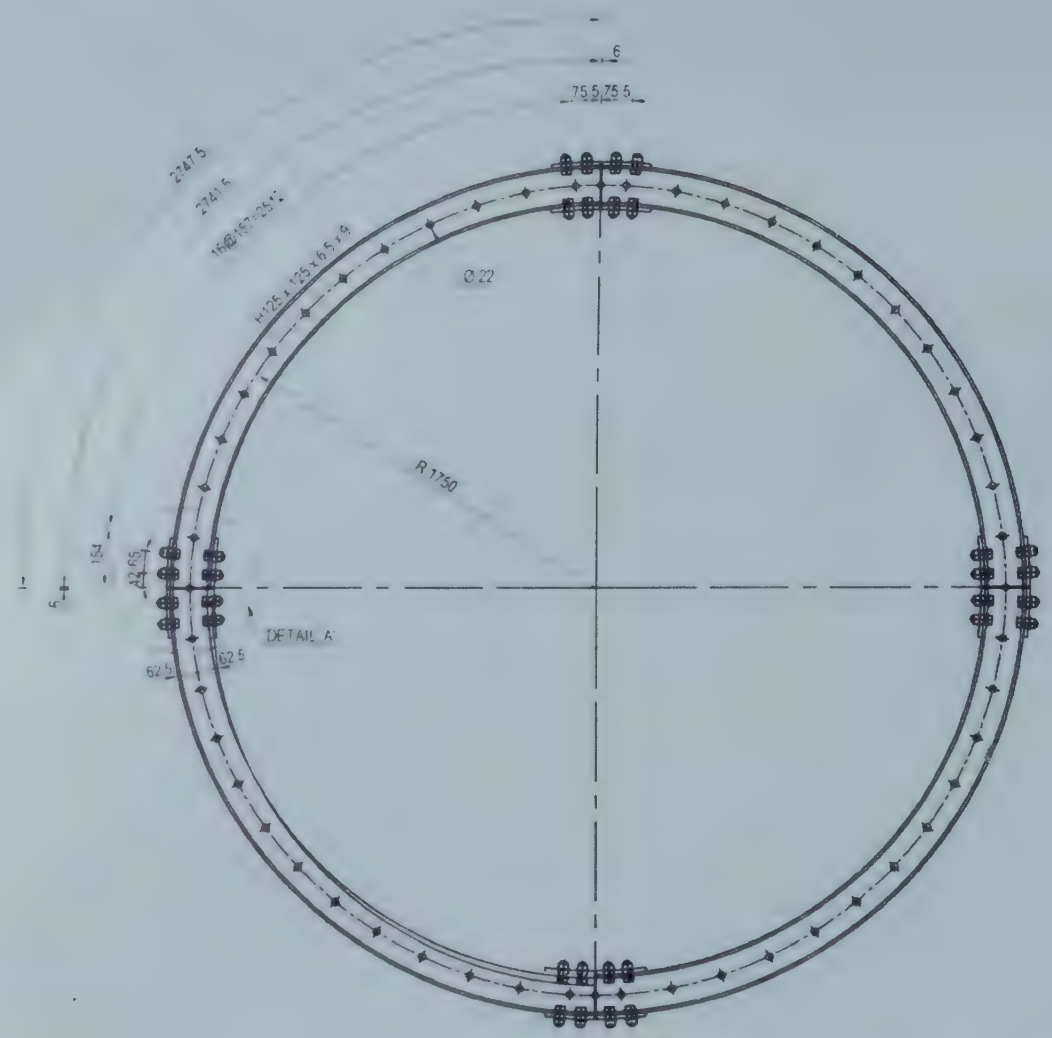
EXPANSION PLAN OF LADDER
SCALE 1:100

SPECIFICATIONS AND QUANTITIES OF DRAINAGE WELL (Ø 3500xH11000+1000)

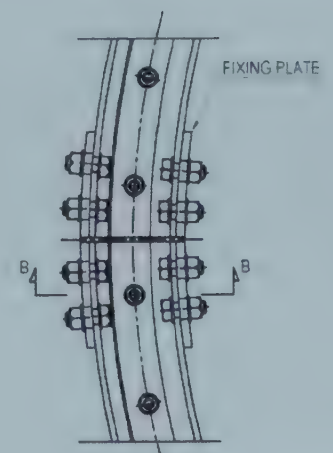
ITEM	SIZE (mm)	UNIT WEIGHT (kg)	QUANTITY	TOTAL WEIGHT (kg)	REMARKS
LINER PLATE	P-10 t=2.7	27.100	126	3414.6	GALVANIZED
LINER PLATE	P-10 t=2.7	27.100	42	1138.2	GALVANIZED Ø 50 (2 LOCATIONS)
ASSEMBLY BOLT	M16 x 30 (4.6)	0.137	2072	283.9	GALVANIZED
ASSEMBLY BOLT	M16 x 45 (8.8 , INCLUDING BOLTS USE FOR RING)	0.158	210	33.2	GALVANIZED
	SUB TOTAL			4869.9	
REINFORCEMENT RING	(L = 2747.5 x 4) 1 RING				
	H 125 x 125 x 6.5 x 9 x 10990	263.000	3	789.0	GALVANIZED
FIXING PLATE	PL 125 x 12 x 310	3.690	24	88.6	GALVANIZED
FIXING BOLT	M20 x 50 (8.8)	0.275	192	52.8	GALVANIZED
SPRING WASHER	M20 USE FOR BOLT		96		
	SUB TOTAL			930.4	
N.T - A TYPE	h:1500	35.500	2	71.0	GALVANIZED
N.T - B TYPE	h:1000	24.500	4	98.0	GALVANIZED
N.T - C TYPE	h:2700	49.600	1	49.6	GALVANIZED
N.T - D1 TYPE	h:2000 (EFFECTIVE LENGTH 1500)	55.300	1	55.3	GALVANIZED
MOUNTING BRACKET (L P PART)	PL 50 x 9 x 200	0.717	20	14.3	GALVANIZED
MOUNTING WASHER (L P PART)	40A x 65	0.262	20	5.2	GALVANIZED
MOUNTING BOLT (L P PART)	M16 x 115 (4.6)	0.270	20	5.4	GALVANIZED
MOUNTING BOLT (L P PART)	M16 x 45 (8.8)	0.158	20	3.2	GALVANIZED
MOUNTING BRACKET (H R PART)	PL 95 x 60 x 9 x 50	0.505	5	2.5	GALVANIZED
MOUNTING WASHER (H R PART)	40A x 124	0.503	5	2.5	GALVANIZED
MOUNTING BOLT (H R PART)	M16 x 175 (4.6)	0.366	5	1.8	GALVANIZED
	SUB TOTAL			308.6	
COVER PLATE	Ø 3650 CHECKERED STEEL PLATE	547.000	1	547.0	GALVANIZED
COVER PLATE CONNECTION BOLT	M16 x 70 (4.6)	-	2	-	GALVANIZED
ASSEMBLY BOLT	M16 x 45 (8.8)	-	6	-	GALVANIZED
TOTAL				6856.1 kg	

NOTE
1 ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED
2 THE QUANTITIES GIVEN IN THE TABLE ARE ONLY FOR REFERENCE USE

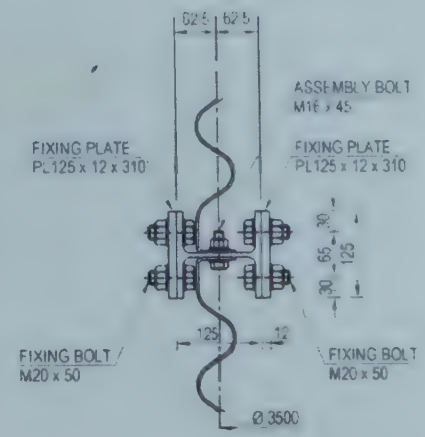
A03-101



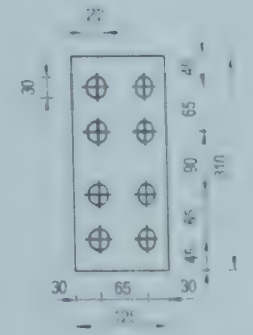
REINFORCEMENT RING
SCALE 1:30



DETAIL 'A'

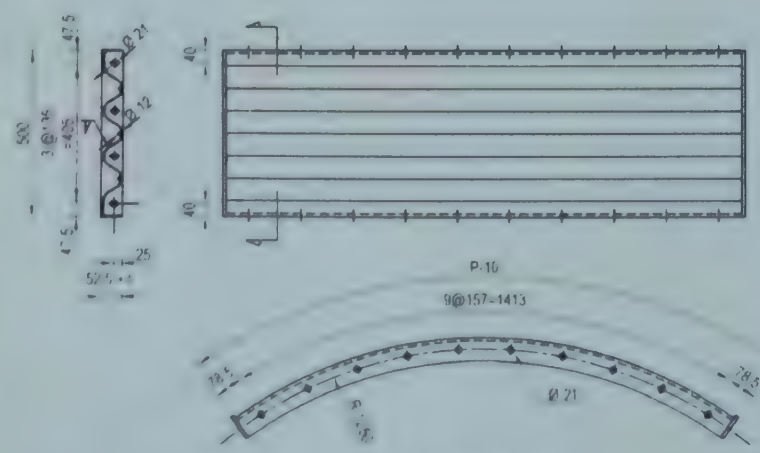


SECTION B - B



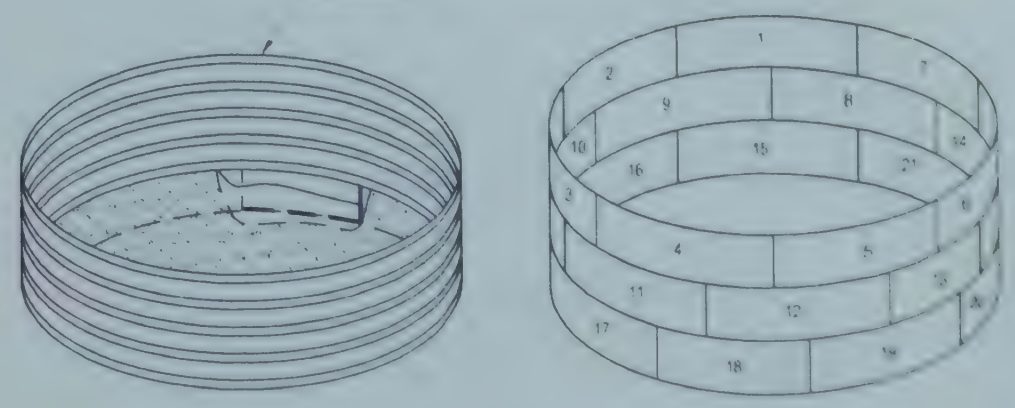
DETAILS OF FIXING PLATE

FIXING DETAILS OF REINFORCEMENT RING
SCALE 1:10



LINER PLATE
SCALE 1:20

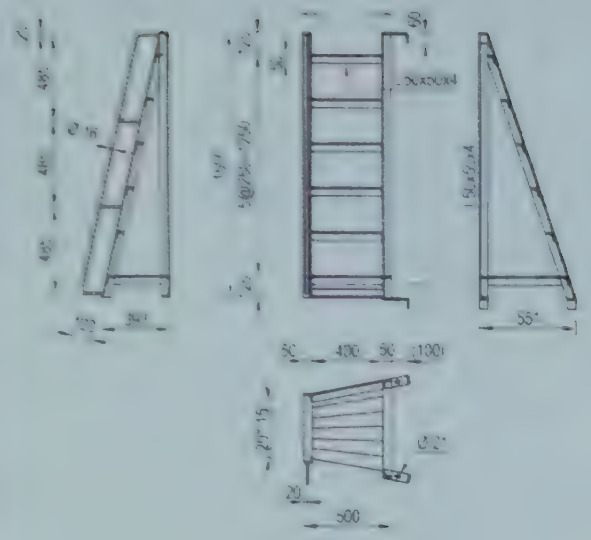
LINER PLATE



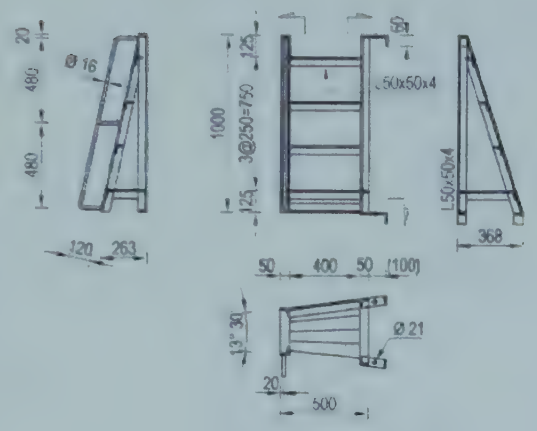
FIXING DETAILS OF LINER PLATE
SCALE 1:40

NOTE
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED

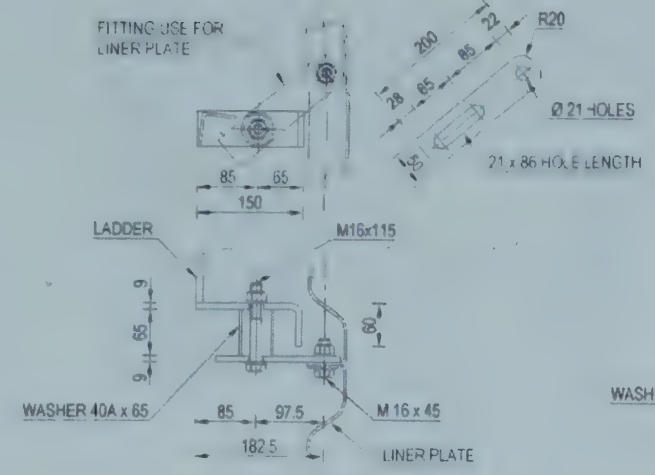
A03-102



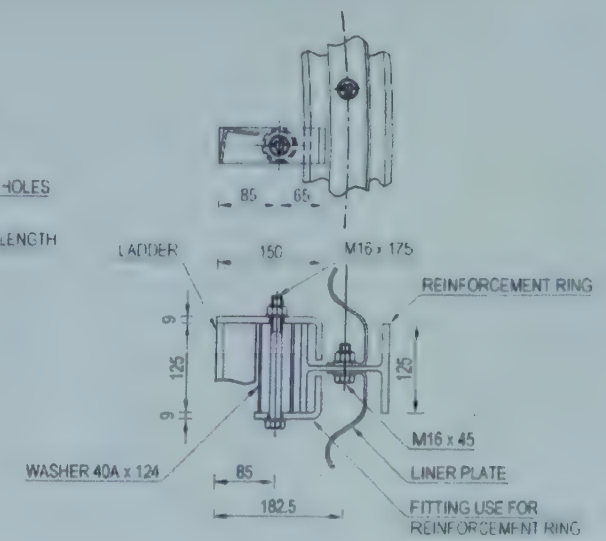
NT - A TYPE
SCALE 1:40



NT - B TYPE
SCALE 1:40

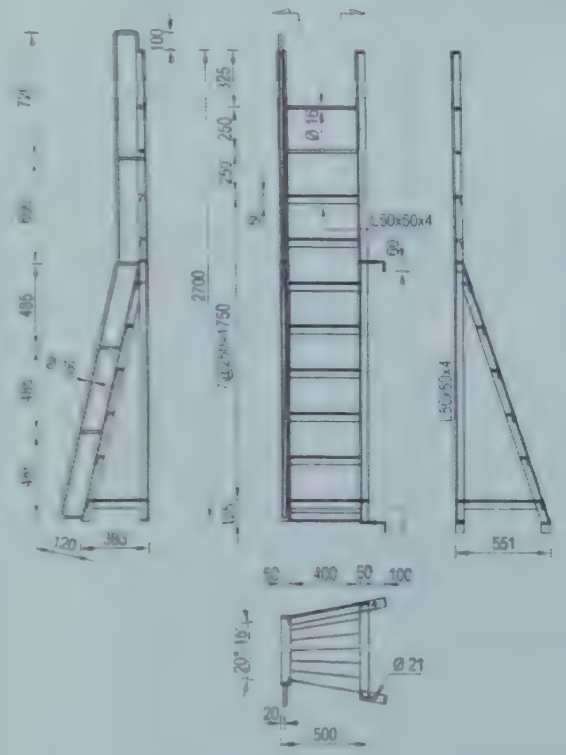


LINER PLATE

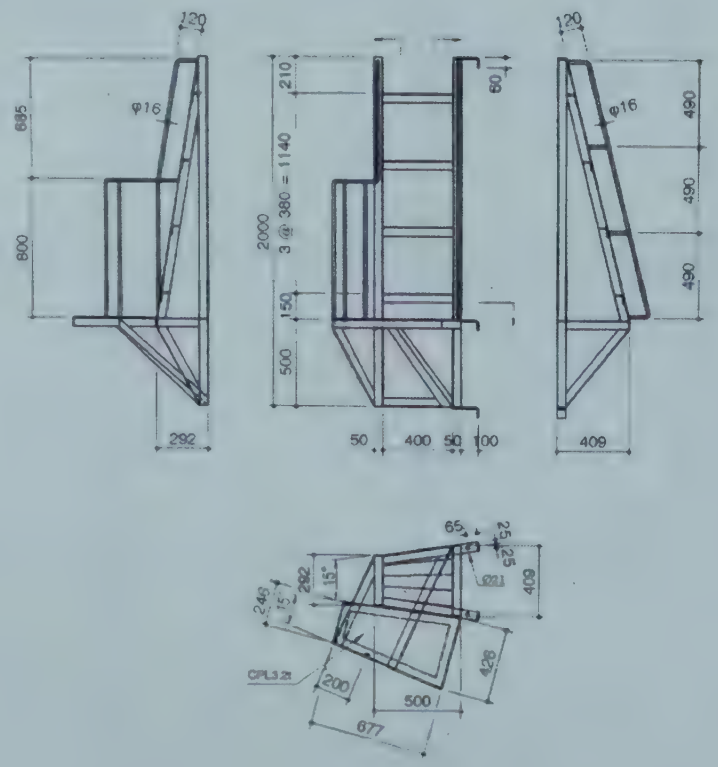


REINFORCEMENT RING

DETAIL OF LADDER
SCALE 1:10



NT - C TYPE
SCALE 1:40



NT - D1 TYPE
SCALE 1:40

MOUNTING MATERIALS OF LINER PLATE

ITEM	DIMENSION	UNIT MASS (Kg./Nos.)
MOUNTING BRACKET	PL 50 x 9 x 200	0.717
WASHER	40A x 65	0.262
MOUNTING BOLT	M16 x 115	0.270
MOUNTING BOLT	M16 x 45	0.158
		1.41

MOUNTING MATERIALS OF REINFORCEMENT RING

ITEM	DIMENSION	UNIT MASS (Kg./Nos.)
MOUNTING BRACKET	PL 99 x 60 x 9 x 50	0.505
WASHER	40A x 124	0.503
MOUNTING BOLT	M16 x 175	0.368
		1.37

SPECIFICATIONS AND QUANTITIES OF LADDER

	LADDER WEIGHT		NT - A TYPE		NT - B TYPE		NT - C TYPE		NT - D1 TYPE	
ITEM	DIMENSION	UNIT MASS	QUANTITY	UNIT MASS	QUANTITY	UNIT MASS	QUANTITY	UNIT MASS	QUANTITY	UNIT MASS
LADDER MATERIAL	L 50 x 50 x 4	3.15 Kg/m	9587 mm	30.2 Kg	6413 mm	20.2 Kg	12787 mm	40.3 Kg	11879 mm	37.4 kg
HANDRAIL	Ø 16 ROUND STEEL BAR	1.60 Kg/m	1972 mm	3.2 Kg	1344 mm	2.2 kg	4521 mm	7.2 Kg	3648 mm	5.8 kg
LANDING	PL 3.2 STEEL CHECKERED PLATE	27.7 Kg/m^2	-		-		-		0.337 m^2	9.3 kg
MOUNTING BRACKET	PL 150 x 60 x 9 x 50	0.700 Kg/Nos.	3 Nos.	2.1 Kg	3 Nos.	2.1 Kg	3 Nos.	2.1 Kg	4 Nos.	2.8 Kg
				35.5 Kg		24.5 Kg		49.6 Kg		55.3 Kg

NOTE

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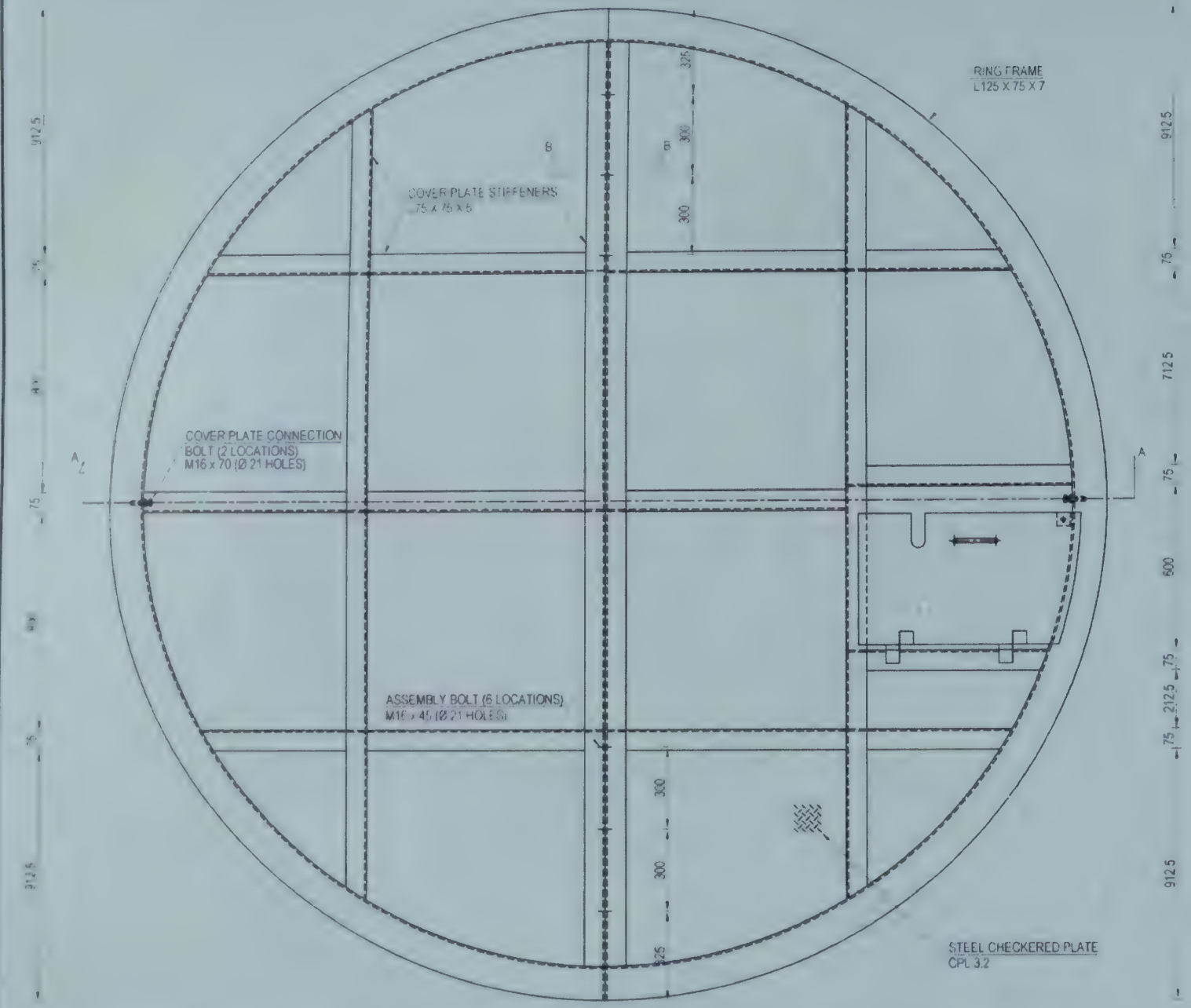
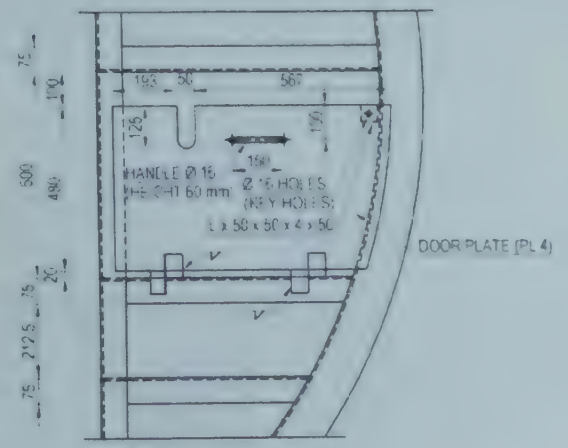
A03-103

A03-104

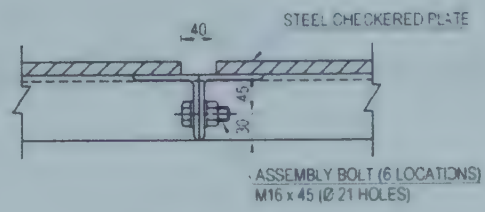


COVER PLATE (DETACHABLE)
SCALE 1:20

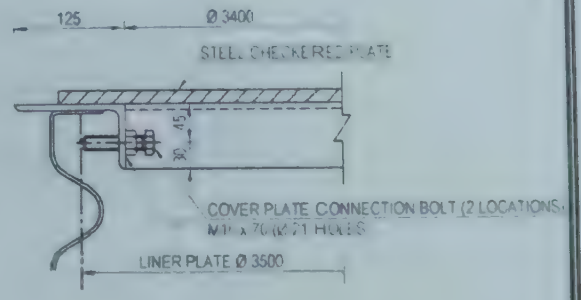
SECTION A - A



DETAIL OF HATCH
SCALE 1:20



SECTION B - B
SCALE 1:8



DETAIL 'C'
SCALE 1:8

ITEM	DIMENSION (mm)	LENGTH, AREA (mm) , (m^2)	UNIT WEIGHT (kg / m) , (kg / m^2)	WEIGHT PER ITEM (kg)	QUANTITY	TOTAL WEIGHT (kg)	REMARKS
RING FRAME	L 125 x 75 x 7	5470.000	10.90	59.600	2	119.2	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	3400.000	6.99	23.800	2	47.6	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	2915.000	6.99	20.400	2	40.8	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	869.000	6.99	6.070	6	36.4	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	599.000	6.99	4.190	4	16.8	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	728.000	6.99	5.090	1	5.1	GALVANIZED
COVER PLATE STIFFENERS	L 75 x 75 x 6	819.000	6.99	5.720	2	11.4	GALVANIZED
STEEL CHECKERED PLATE	CPL 3.2	4.880	27.69	135.000	1	135.0	GALVANIZED
STEEL CHECKERED PLATE	CPL 3.2	4.328	27.69	120.000	1	120.0	GALVANIZED
DOOR PLATE	PL 4			11.900	1	11.9	GALVANIZED
FLAG HINGE	4.5 x 105 x 115			0.450	2	0.9	GALVANIZED
HANDLE	Ø 16			0.432	1	0.4	GALVANIZED
LATCH	L 50 x 50 x 4			0.158	1	0.2	GALVANIZED
COVER PLATE CONNECTION BOLT	M16 (4.6)	70.000		0.199	2	0.4	GALVANIZED
ASSEMBLY BOLT	M16 (8.8)	45.000		0.158	6	0.9	GALVANIZED
TOTAL						547.0 kg	

NOTE

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
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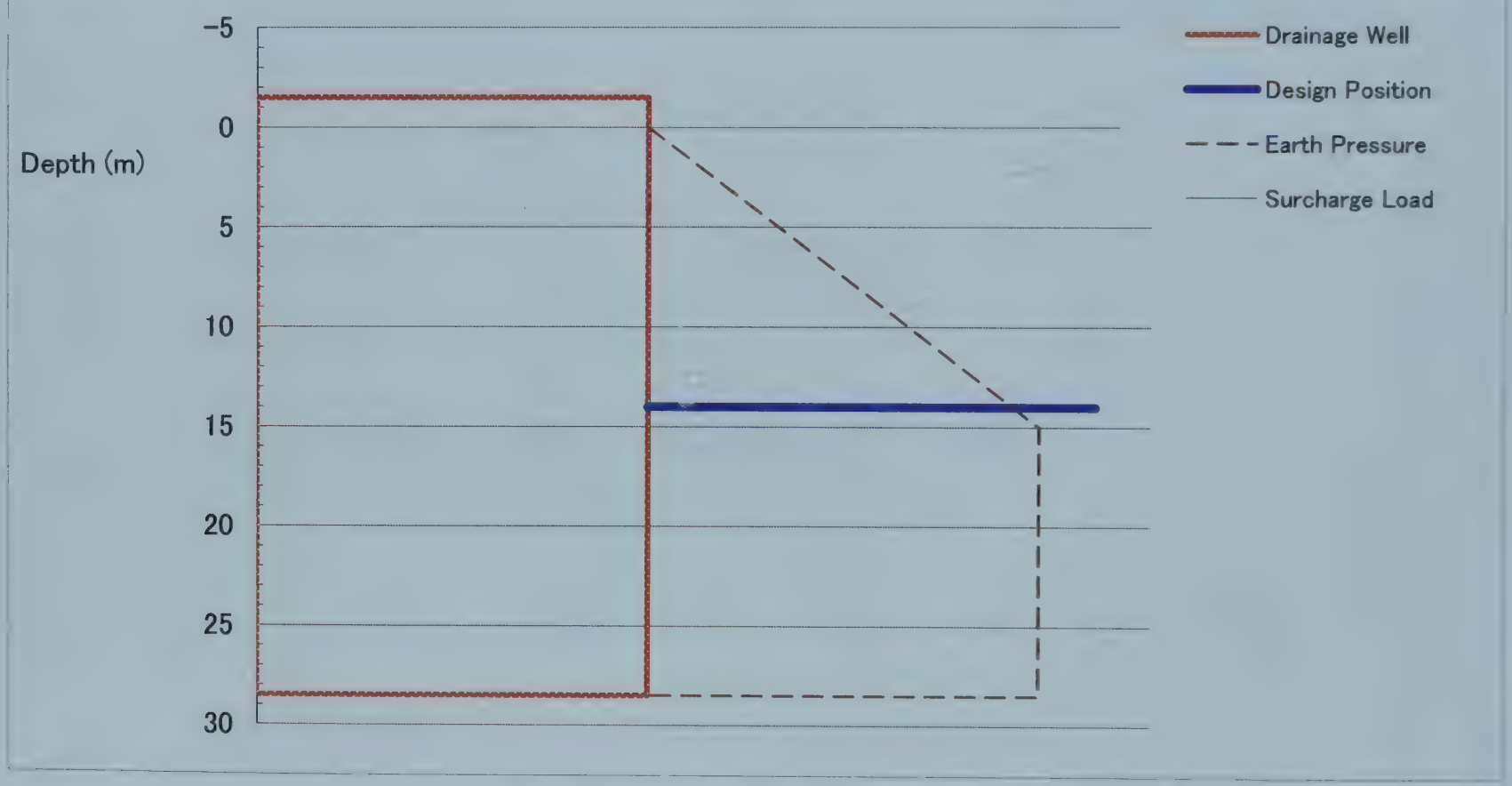
Appendix 8

Calculation for Drainage Well on L2

Drainage Well Design Conditions

Cross-Section Design	Design Position G.L.-	h	14	m
Structural Conditions	Diameter of Drainage Well	D	3.5	m
	Total Length of Drainage Well	H	30	m
	Depth of Drainage Well G.L.-	h_0	28.5	m
	Thickness of Liner Plate	t	2.7	mm
	Cross-Section Loss Coefficient of Liner Plate	ω	0.8	(ω=0.8)
	Reinforcing Ring Interval	L_H	2	m
	Reinforcing Ring Component H ×B×t1×t2		125×125×6.5×9	mm
Soil Conditions	Unit Volume Weight of Soil	γ_t	20	kN/m^3
Load	Surcharge Load	q	0	kN/m^2
Earth Pressure (Equation of Earth Pressure at Rest)	Earth Pressure Intensity Fixed Depth G.L.-	h_f	15	m (15m～)
	Coefficient of Earth Pressure at Rest	k	0.5	(k=0.5)
Coefficient	Modulus of Elasticity of Steel	E	200	kN/mm^2
Allowable Stress	Liner Plate	σ_{La}	120	N/mm^2
	Reinforcing Ring	σ_{Ha}	140	N/mm^2
Verification of Buckling	Safety Factor	f	1.5	(f=1.5～2)



Liner Plate (Per 1m)

Plate Thickness t (mm)	Cross-Sectional Area A_{LO} (cm^2/m)	Section Modulus Z_{LO} (cm^3/m)	Sectional Secondary Moment I_{LO} (cm^4/m)	Effective Cross-Sectional Area $A_L=A_{LO}\times\omega$ (cm^2/m)	Effective Section Modulus $Z_L=Z_{LO}\times\omega$ (cm^3/m)	Effective Sectional Secondary Moment $I_L=I_{LO}\times\omega$ (cm^4/m)
2.7	39.76	46.00	141.00	31.81	36.80	112.80

Reinforcing Ring

Reinforcing Ring Interval	Steel	Per Location			Per 1m		
		Cross-Sectional Area	Section Modulus	Sectional Secondary Moment	Cross-Sectional Area	Section Modulus	Sectional Secondary Moment
L_H (m)	$H\times B\times t_1\times t_2$ (mm)	A_{HO} (cm^2/location)	Z_{HO} (cm^3/location)	I_{HO} (cm^4/location)	A_H (cm^2/m)	Z_H (cm^3/m)	I_H (cm^4/m)
2.0	125×125×6.5×9	30.00	134.00	839.00	15.00	67.00	419.50

Liner Plate + Reinforcing Ring

Axial Force Distribution Rate (Cross-Sectional Area Ratio)		Bending Moment Distribution Rate (Sectional Secondary Moment Ratio)		Original Deflection	Allowable Buckling Load	Buckling Limit Load
Liner Plate	Reinforcing Ring	Liner Plate	Reinforcing Ring			
α_L	α_H	β_L	β_H	$\delta_0=0.01\times R$ (m)	q_a (kN/m^2)	q_k (kN/m^2)
0.680	0.320	0.212	0.788	0.0175	397.29	595.93

Drainage Well Design (In Combination with Reinforcing Ring)

Design Earth Pressure		Axial Compressive Force	Verification of Buckling	Maximum Bending Moment	Verification of Stress				Overall Assessment
					Liner Plate		Reinforcing Ring		
Design Position h (m)	Earth Pressure Intensity P_h (kN/m^2)	N (kN/m)	Assessment $P_h\leqq q_a$	M_{max} (kN·m/m)	Compressive Stress σ_L (N/mm^2)	Assessment $\sigma_L\leqq\sigma_{La}$	Compressive Stress σ_H (N/mm^2)	Assessment $\sigma_H\leqq\sigma_{Ha}$	
14.000	140.00	245.00	OK	5.60	85	OK	118	OK	OK
0.500	5.00	8.75	OK	0.15	3	OK	4	OK	OK
1.000	10.00	17.50	OK	0.31	6	OK	7	OK	OK
1.500	15.00	26.25	OK	0.47	8	OK	11	OK	OK
2.000	20.00	35.00	OK	0.63	11	OK	15	OK	OK
2.500	25.00	43.75	OK	0.80	14	OK	19	OK	OK
3.000	30.00	52.50	OK	0.97	17	OK	23	OK	OK
3.500	35.00	61.25	OK	1.14	20	OK	26	OK	OK
4.000	40.00	70.00	OK	1.31	23	OK	30	OK	OK
4.500	45.00	78.75	OK	1.49	25	OK	34	OK	OK
5.000	50.00	87.50	OK	1.67	28	OK	38	OK	OK
5.500	55.00	96.25	OK	1.86	31	OK	42	OK	OK
6.000	60.00	105.00	OK	2.04	34	OK	46	OK	OK
6.500	65.00	113.75	OK	2.23	37	OK	51	OK	OK
7.000	70.00	122.50	OK	2.43	40	OK	55	OK	OK
7.500	75.00	131.25	OK	2.63	43	OK	59	OK	OK
8.000	80.00	140.00	OK	2.83	46	OK	63	OK	OK
8.500	85.00	148.75	OK	3.04	49	OK	67	OK	OK
9.000	90.00	157.50	OK	3.25	52	OK	72	OK	OK
9.500	95.00	166.25	OK	3.46	55	OK	76	OK	OK
10.000	100.00	175.00	OK	3.68	59	OK	81	OK	OK
10.500	105.00	183.75	OK	3.90	62	OK	85	OK	OK
11.000	110.00	192.50	OK	4.13	65	OK	90	OK	OK
11.500	115.00	201.25	OK	4.36	68	OK	94	OK	OK
12.000	120.00	210.00	OK	4.60	71	OK	99	OK	OK
12.500	125.00	218.75	OK	4.84	75	OK	104	OK	OK
13.000	130.00	227.50	OK	5.09	78	OK	108	OK	OK
13.500	135.00	236.25	OK	5.35	81	OK	113	OK	OK
14.000	140.00	245.00	OK	5.60	85	OK	118	OK	OK
14.500	145.00	253.75	OK	5.87	88	OK	123	OK	OK
15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
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15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK

15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
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15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK
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15.000	150.00	262.50	OK	6.14	91	OK	128	OK	OK

Drainage Well Design (In Combination with Reinforcing Ring)

Design calculations are to be carried out in accordance with Design Volume II of the Ministry of Construction Technical Criteria for River Works and the Liner Plate Design and Construction Manual (June 2000).

(1)Design Conditions

1)Cross-Section Design

Design Position G.L.-	h=	14.000	(m)

2)Structural Conditions

Diameter of Drainage Well	D=	3.5	(m)
Total Length of Drainage Well	H=	30	(m)
Depth of Drainage Well G.L.-	h_0=	28.5	(m)
Plate Thickness	t=	2.7	(mm)
Cross-Section Loss Coefficient	ω=	0.8	
Reinforcing Ring Interval	L_H=	2	(m)
Reinforcing Ring Component ($H\times B\times t_1\times t_2$)		$125\times125\times6.5\times9$	

3)Soil Conditions

Unit Volume Weight of Soil	γ_t=	20	(kN/m^3)

4)Load

Surcharge Load	q=	0	(kN/m^2)

5)Earth Pressure (Equation for Earth Pressure At Rest)

Earth Pressure Intensity Fixed Depth G.L.-	h_f=	15	(m)
Coefficient of Earth Pressure at Rest	k=	0.5	

6)Coefficient

Modulus of Elasticity of Steel	E=	200	(kN/mm^2)

7)Allowable Stress

Liner Plate	σ_{La}=	120	(N/mm^2)
Reinforcing Ring	σ_{Ha}=	140	(N/mm^2)

8)Verification of Buckling

Safety Factor	f=	1.5	

(2)Design Earth Pressure

Calculation of Earth Pressure Intensity P_h

Design Position	h=	14	(m)	;when $h\geqq h_f$, $h=h_f$
Unit Volume Weight of Soil	γ_t=	20	(kN/m^3)	
Surcharge Load	q=	0	(kN/m^2)	
Coefficient of Earth Pressure At Rest	k=	0.5		

$P_h = k\times(\gamma_t\times h+q)$

$= 0.5 \times (20 \times 14 + 0)$

$= 140.00$ (kN/m^2)

(3) Verification of Buckling

Calculation of Buckling Limit Load q_k

Modulus of Elasticity of Steel	$E=$	200	(kN/mm^2)
Sectional Secondary Moment of Liner Plate	$I_{LO}=$	141.00	(cm^4/m)
Cross-Section Loss Coefficient of Liner Plate	$\omega=$	0.8	
Effective Sectional Secondary Moment of Liner Plate	$I_L=$	112.80	(cm^4/m) ; $I_L=I_{LO}\times\omega$
Sectional Secondary Moment of Reinforcing Ring	$I_{HO}=$	839.00	$(cm^4/location)$
Reinforcing Ring Interval	$L_H=$	2.0	(m)
Sectional Secondary Moment of Reinforcing Ring (Per 1m)	$I_H=$	419.50	(cm^4/m) ; $I_H=I_{HO}/L_H$
Radius of Drainage Well	$R=$	1.750	(m)

$$q_k = \frac{3\times E\times (I_L+I_H)}{R^3}$$

$$= \frac{3 \times 200 \times 10^6 \times (112.80 + 419.50) \times 10^{-8}}{1.750^3}$$

$$= 595.93 \ (kN/m^2)$$

Calculation of Allowable Buckling Load q_a

Modulus of Elasticity of Steel	$E=$	200	(kN/mm^2)
Sectional Secondary Moment of Liner Plate	$I_{LO}=$	141.00	(cm^4/m)
Cross-Section Loss Coefficient of Liner Plate	$\omega=$	0.8	
Effective Sectional Secondary Moment of Liner Plate	$I_L=$	112.80	(cm^4/m) ; $I_L=I_{LO}\times\omega$
Sectional Secondary Moment of Reinforcing Ring	$I_{HO}=$	839.00	$(cm^4/location)$
Reinforcing Ring Interval	$L_H=$	2.0	(m)
Sectional Secondary Moment of Reinforcing Ring (Per 1m)	$I_H=$	419.50	(cm^4/m) ; $I_H=I_{HO}/L_H$
Radius of Drainage Well	$R=$	1.750	(m)
Safety Factor	$f=$	1.5	

$$q_a = \frac{3\times E\times (I_L+I_H)}{f\times R^3} \geqq P_h$$

$$= \frac{3 \times 200 \times 10^6 \times (112.80 + 419.50) \times 10^{-8}}{1.5 \times 1.750^3}$$

$$= 397.29 \ (kN/m^2) \geqq 140.00 \ (kN/m^2) \rightarrow OK$$

(4)Verification of Stress

Calculation of Axial Compressive Force N

Earth Pressure Intensity	P_h=	140.00	(kN/m^2)
Radius of Drainage Well	R=	1.750	(m)

$N = P_h \times R$

$= 140.00 \times 1.750$

$= 245.00$ (kN/m)

Calculation of Maximum Bending Moment M_{max}

Axial Compressive Force	N=	245.00	(kN/m)	
Earth Pressure Intensity	P_h=	140.00	(kN/m^2)	
Radius of Drainage Well	R=	1.750	(m)	
Original Deflection	δ_0=	0.0175	(m)	; $\delta_0=0.01\times R$
Buckling Limit Load	q_k=	595.93	(kN/m^2)	

$$M_{max} = \left| N \times \frac{\delta_0}{1-P_h/q_k} \right|$$

$$= \left| 245.00 \times \frac{0.0175}{1 - 140.00 / 595.93} \right|$$

$= 5.60$ (kN·m/m)

Calculation of Axial Force Distribution Rate of Liner Plate α_L

Cross-Sectional Area of Liner Plate	A_{LO}=	39.76	(cm^2/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Cross-Sectional Area of Liner Plate	A_L=	31.81	(cm^2/m)	; $A_L=A_{LO}\times\omega$
Cross-Sectional Area of Reinforcing Ring	A_{HO}=	30.00	(cm^2/location)	
Reinforcing Ring Interval	L_H=	2.0	(m)	
Cross-Sectional Area of Reinforcing Ring (Per 1m)	A_H=	15.00	(cm^2/m)	; $A_H=A_{HO}/L_H$

$\alpha_L = A_L/(A_L+A_H)$

$= 31.81 / (31.81 + 15.00)$

$= 0.680$

Calculation of Bending Moment Distribution Rate of Liner Plate β_L

Sectional Secondary Moment of Liner Plate	I_{LO}=	141.00	(cm^4/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Sectional Secondary Moment of Liner Plate	I_L=	112.80	(cm^4/m)	; $I_L=I_{LO}\times\omega$
Sectional Secondary Moment of Reinforcing Ring	I_{HO}=	839.00	(cm^4/location)	
Reinforcing Ring Interval	L_H=	2.0	(m)	
Sectional Secondary Moment of Reinforcing Ring (Per 1m)	I_H=	419.50	(cm^4/m)	; $I_H=I_{HO}/L_H$

$\beta_L = I_L/(I_L+I_H)$

$= 112.80 / (112.80 + 419.50)$

$= 0.212$

Calculation of Compressive Stress Generated in Liner Plate σ_L

Axial Force Distribution Rate of Liner Plate	α_L=	0.680		
Axial Compressive Force	N=	245.00	(kN/m)	
Cross-Sectional Area of Liner Plate	A_{L0}=	39.76	(cm^2/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Cross-Sectional Area of Liner Plate	A_L=	31.81	(cm^2/m)	; $A_L=A_{L0}\times\omega$
Bending Moment Distribution Rate of Liner Plate	β_L=	0.212		
Maximum Bending Moment	M_{max}=	5.60	(kN·m/m)	
Section Modulus of Liner Plate	Z_{L0}=	46.00	(cm^3/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Section Modulus of Liner Plate	Z_L=	36.80	(cm^3/m)	; $Z_L=Z_{L0}\times\omega$

$$\sigma_L = \frac{\alpha_L \times N}{A_L} + \frac{\beta_L \times M_{max}}{Z_L} \leqq \sigma_{La}$$

$$= \frac{0.680 \times 245.00 \times 10^3}{31.81 \times 10^2} + \frac{0.212 \times 5.60 \times 10^6}{36.80 \times 10^3}$$

$$= 85\ (N/mm^2) \leqq 120\ (N/mm^2) \rightarrow OK$$

Calculation of Axial Force Distribution Rate of Reinforcing Ring α_H

Cross-Sectional Area of Liner Plate	A_{L0}=	39.76	(cm^2/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Cross-Sectional Area of Liner Plate	A_L=	31.81	(cm^2/m)	; $A_L=A_{L0}\times\omega$
Cross-Sectional Area of Reinforcing Ring	A_{H0}=	30.00	(cm^2/location)	
Reinforcing Ring Interval	L_H=	2.0	(m)	
Cross-Sectional Area of Reinforcing Ring (Per 1m)	A_H=	15.00	(cm^2/m)	; $A_H=A_{H0}/L_H$

$$\alpha_H = A_H/(A_L+A_H)$$

$$= 15.00 / (31.81 + 15.00)$$

$$= 0.320$$

Calculation of Bending Moment Distribution Rate of Reinforcing Ring β_H

Sectional Secondary Moment of Liner Plate	I_{LO}=	141.00	(cm^4/m)	
Cross-Section Loss Coefficient of Liner Plate	ω=	0.8		
Effective Sectional Secondary Moment of Liner Plate	I_L=	112.80	(cm^4/m)	; $I_L=I_{LO}\times\omega$
Sectional Secondary Moment of Reinforcing Ring	I_{HO}=	839.00	(cm^4/location)	
Reinforcing Ring Interval	L_H=	2.0	(m)	
Sectional Secondary Moment of Reinforcing Ring (Per 1m)	I_H=	419.50	(cm^4/m)	; $I_H=I_{HO}/L_H$

$$\beta_H = I_H/(I_L+I_H)$$

$$= 419.50 / (112.80 + 419.50)$$

$$= 0.788$$

Calculation of Compressive Stress Generated in Reinforcing Ring σ_H

Axial Force Distribution Rate of Reinforcing Ring	α_H=	0.320		
Axial Compressive Force	N=	245.00	(kN/m)	
Cross-Sectional Area of Reinforcing Ring	A_{HO}=	30.00	(cm^2/location)	
Reinforcing Ring Interval	L_H=	2.0	(m)	
Cross-Sectional Area of Reinforcing Ring (Per 1m)	A_H=	15.00	(cm^2/m)	; $A_H=A_{HO}/L_H$
Bending Moment Distribution Rate of Reinforcing Ring	β_H=	0.788		
Maximum Bending Moment	M_{max}=	5.60	(kN·m/m)	
Section Modulus of Reinforcing Ring	Z_{HO}=	134.00	(cm^3/location)	
Section Modulus of Reinforcing Ring (Per 1m)	Z_H=	67.00	(cm^3/m)	; $Z_H=Z_{HO}/L_H$

$$\sigma_H = \frac{\alpha_H\times N}{A_H} + \frac{\beta_H\times M_{max}}{Z_H} \leqq \sigma_{Ha}$$

$$= \frac{0.320 \times 245.00 \times 10^3}{15.00 \times 10^2} + \frac{0.788 \times 5.60 \times 10^6}{67.00 \times 10^3}$$

$$= 118\ (N/mm^2) \leqq 140\ (N/mm^2) \rightarrow OK$$

Liner Plate Component (Per 1m)　　Cross-Section Loss Coefficient $\omega = 0.8$

No.	Plate Thickness t (mm)	Cross-Sectional Area A_{LO} (cm^2/m)	Section Modulus Z_{LO} (cm^3/m)	Sectional Secondary Moment I_{LO} (cm^4/m)	Effective Cross-Sectional Area $A_L=A_{LO}\times\omega$ (cm^2/m)	Effective Section Modulus $Z_L=Z_{LO}\times\omega$ (cm^3/m)	Effective Sectional Secondary Moment $I_L=I_{LO}\times\omega$ (cm^4/m)
1	2.7	39.76	46.00	141.00	31.81	36.80	112.80
2	3.2	47.12	54.40	168.00	37.70	43.52	134.40
3	4.0	58.86	67.40	210.00	47.09	53.92	168.00
4	4.5	66.22	75.80	238.00	52.98	60.64	190.40
5	5.3	77.90	88.80	280.00	62.32	71.04	224.00
6	6.0	88.20	100.00	320.00	70.56	80.00	256.00
7	7.0	102.90	116.00	376.00	82.32	92.80	300.80

Reinforcing Ring Component

No.	Steel $H\times B\times t_1\times t_2$ (mm)	Cross-Sectional Area A_{HO} (cm^2)	Section Modulus Z_{HO} (cm^3)	Sectional Secondary Moment I_{HO} (cm^4)
1	100×100×6×8	21.59	75.60	378.00
2	125×125×6.5×9	30.00	134.00	839.00
3	150×150×7×10	39.65	216.00	1,620.00
4	175×175×7.5×11	51.42	331.00	2,900.00
5	200×200×8×12	63.53	472.00	4,720.00
6	250×250×9×14	91.43	860.00	10,700.00
7	300×300×10×15	118.40	1,350.00	20,200.00
8	350×350×12×19	171.90	2,280.00	39,800.00
9	400×400×13×21	218.70	3,330.00	66,600.00

No.	Reinforcing Ring Interval (m)
1	Not set
2	2.0
3	1.5
4	1.0
5	0.5

Appendix IV

Recommendation on Countermeasure Design: NH717

Recommendations on Countermeasures against Gravity Deformation on NH717 in the State of West Bengal

September 2021

JICA Experts for Capacity Development Project on Highways in Mountainous Regions

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Appendix1: Requirements on TOR

Appelndix2: Plan View for Survey

Appendix3: Borehole Record Analysis

Appendix4: Groundwater Variation Analysis Charts (GVAC)

Appendix5: Slice Reading

Appendix6: Image of Control Works

1 Introduction

As a part of Pilot Projects, this report summarizes technical recommendation on countermeasures against gravity deformation at the pilot project site on NH717 in the State of West Bengal.

At the site, alternative options for rehabilitation of NH717 are considered; detour by a tunnel or a bridge or simply introducing slope protection while using the current alignment. For either case, the gravity deformation blocks which are affecting the section should be firstly stabilized / settled, otherwise an alternative option should be negatively affected and eventually demolished in longer term.

In this recommendation, the properties of the gravity deformation blocks affecting the section are estimated and then countermeasures to stabilize / settle gravity deformation are examined through slope stability analysis, while leaving remarks on implementation.

We expect this recommendation would help implementation of permanent countermeasures against gravity deformation blocks at the site.

For further understanding of gravity deformation, please refer to Chapter 1.1.3, 1.1.4, 1.2.2, 1.2.3, 1.3.2, 1.5.3, and 2.6 of JICA Slope Guideline.

2 Available Data for Recommendation

Prior to this recommendation, we asked NHIDCL to obtain necessary data by hiring a local consultant. Our requests were incorporated into TOR for hiring a DPR consultant. The excerpted TOR is shown in Appendix-1.

The requirements are essential for estimating properties of the gravity deformation blocks and considering countermeasure options. Thus these are ordinary and common survey items in Japan.

In India, however, considering capabilities of local consultants and difficulties of the site, some of the TOR might be challenging. For some survey items, the results failed to meet the requirements and inevitably affect quality of this recommendation.

The reasons of failing to meet the requirements are partially due to the delay of the study caused by COVID-19 and the difficulties from severe weather and topographic conditions of

the site. Considering the fact that such survey items are ordinary in Japan, however, the difference in survey technologies between Japan and India should lie largely behind the failing requirements. By improving the technologies of local consultants for such items that failed to meet the requirements, we think the technologies for survey and investigation of landslide and gravity deformation should be drastically improved in India.

2.1 Core Boring

Core borings were carried out at the 9 sites as planned. The local consultant proposed slight shift of 2 borings. The locations of core borings are shown in Appendix-2: Plan View. In Appendix-2, the blocks of gravity deformation interpreted from topographic maps are also shown with its border in bold black line.

The required core recovery ratio is 95 % or more. In case such core recovery ratio was not expected, using borehole TV (refer to 1.1.2(2)(iv) of JICA Slope Guideline) to directly observe wall of borehole was required. According to the reports by the local consultant, however, the core recovery ratio was substantially low and only hard rock parts seems to have been recovered.

Improving core recovery ratio will directly improve accuracy of survey. Thus efforts to improve core recovery ratio are worth to be paid. In general, core recovery ratio depends on the following factors:

- Quantity of drilling water to be provided to a core bit
- Pressure of drilling water to be provided to a core bit
- Viscosity and density of drilling water
- Rotation speed of drilling
- Pressure to be provided to a core bit, including weights of drilling rods and tools

To improve core recovery ratio, sensitive control of the aforementioned items is essential. In addition, proper maintenance of drilling tools and periodic supply of consumable core bits are necessary. Proper fixing of a drilling rig on site is also important for stable drilling and loading pressure; both are important factors for core recovery ratio.

To stimulate local consultants to improve core recovery ratio, giving incentive for perfectly high core recovery ratio may be an option. In addition, approving additional payment for adding borehole TV may be effective.

2.2 Groundwater Variation Analysis Chart (GVAC)

Organizing groundwater variation analysis chart (GVAC, refer to 1.1.2(2)(v) of JICA Slope Guideline) using daily records of drilling was a part of the requirements. For all 9 boreholes, GVACs were prepared but information of casing depths for each day was insufficient. Organizing GVACs is one of the easiest and most reliable methods to grasp groundwater conditions at the time of boring. Thus preparing daily drilling records each working day with information of drilling work as much as possible including inserted depths of casing is essential and required for the study of landslide and gravity deformation.

2.3 Locations of boreholes on profiles / cross sections.

Initially locations of the boreholes were not shown on the profiles of the survey lines provided from NHIDCL. Thus we requested to show such locations on the profiles and NHIDCL provided those with such locations.

Although it is not required in the TOR, the coordinates of both ends of survey lines as well as survey points (chainage) and borehole locations should be recorded so that survey points / lines and borehole locations are precisely shown on plan views and profiles / cross sections. It is general requirements of quality control of survey results.

2.4 Landslide Monitoring

The results of landslide monitoring using boreholes were not submitted. Landslide monitoring, such as observation of groundwater level and underground displacement, is essential when the local consultant wants to update this recommendation. Thus continuous observation is recommended.

The results of rain gauge observation was submitted in part but did not show significant information.

2.5 Geophysical Exploration

The submitted results of the seismic exploration was effective reference for examining the loosened zones of the gravity deformation blocks.

On the other hand, resistivity survey was done with a different survey method from the requirements. The required survey method is 2-D survey with multiple electrodes aligned on the survey lines extended for several hundred meters, the results of which is shown in the figure above. The submitted results used the vertical electric survey (VES), much simple method compared to the 2-D one.

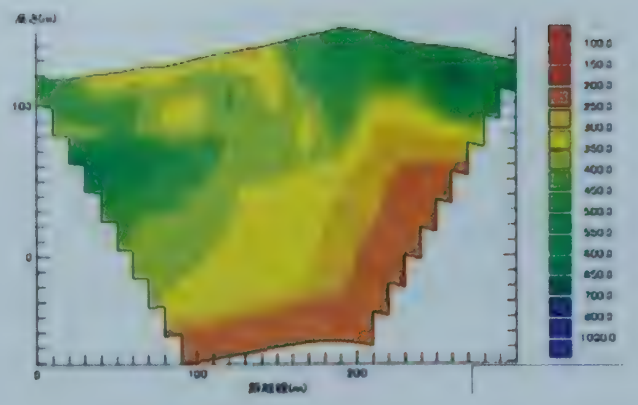


Figure 2-1 Required Results for Resistivity Survey

3 Examination and Analysis

The following steps were made for the recommendation.

1. Estimating depths of loosened zones
2. Estimating gravity deformation blocks (area of blocks and lower ends of loosened zones)
3. Estimating current high water levels
4. Estimating design shear strength parameters
5. Examining groundwater drainage (horizontal drainage borings)
6. Examining earth removal works

3.1 Estimating Depths of Loosened Zones

This step should be done based on the visual inspection of core samples and observation of underground displacement monitoring. Due to the poor core recovery ratio, however, visual inspection of core samples /photos as well as drilling logs cannot provide entire information for whole depth of borehole. In addition, no monitoring results were provided.

Based on daily drilling records, therefore, depths of loosened zones were examined. The results of examination is shown in Appendix-3: Borehole Record Analysis. At the same time, GVAC of each borehole is shown in Appendix-4: Groundwater Variation Analysis.

Drilling speed and leakage ratio are calculated for each drilling run from the daily drilling records in the aim of finding "less loosened zones of bedrock", which is identified by comparing average value of drilling speed and leakage ratio. The depths which show lower values than the average are identified "less loosened" and colored.

The lower ends of loosened zones are decided based on comprehensive analysis of multiple factors shown in Appendix-3.

Reasons for identification of less loosened zones:

Less Drilling Speed: Drilling was made with single core tube. Thus hardness of bedrock is reflected sensitively to drilling speed. In other word, the section where longer drilling time is required, such section is considered as hard rock or less crack section.

Less leakage ratio: Less leakage is considered as less crack and thus less loosened zone as well.

Behavior on GVAC: In case the following features are detected on GVAC, the corresponding section is considered to have less leakage ratio.

A: Groundwater levels before drilling show similar constant values on consecutive working days.

B: Difference between the groundwater levels after drilling and before drilling of the following working day show little difference.

Hard rock sections: The sections identified as "hard rock" on drilling logs or daily working records are considered as "less crack section". In addition, the sections where core samples with measurable lengths are also considered as "less crack section" as well.

3.2 Estimating Gravity Deformation Blocks

The results of examination of loosened zones, which is described in the previous section, and the groundwater levels observed at the end of drilling of each borehole are projected on the profiles of each survey line provided from NHIDCL. The slip surfaces for slope stability analysis on each survey line are decided by connecting the lower ends of loosened zones. At the upper end of such slip surface, the steep slope of the head scarp is extended into subsurface until it intersects with upper extension of the slip surface mentioned above. At the lower end of such slip surface, slope surface just outside of the gravity deformation block is extended into the block until it intersects with lower extension of the slip surface.

For the profile where only one borehole is available, the slip surface is drawn from the lower end of the loosened zone at the available borehole and extended upward and downward by referring to average gradients of slope surface of the corresponding gravity

deformation block.

By smoothly connecting the slip surface decided through the aforementioned procedure and the points where abnormalities in elevation contour are observed, boundaries of gravity deformation blocks are decided.

For the profile Line-2, the direction of survey line does not properly represent the targeted gravity deformation block. Thus Line-2 is not suitable for stability analysis of the corresponding block. Thus a new survey line that suits for such stability analysis was newly established on the drawing roughly parallel to Line-3 and called Projected Line-2. Borehole locations are projected to Projected Line-2 for analysis.

For the profile Line-3, the available borehole for analysis is located outside the targeted gravity deformation block. Thus underground information for analysis was prepared refereeing to the Projected Line-2 including information on slip surface, after no discrepancy is confirmed due to the reference of the Projected Line-2.

3.3 Estimating Current High Water Levels

A High Water Level (HWL) for stability analysis was set refereeing to the groundwater level observed at the end of drilling of each borehole.

The same water level was used as a Critical Water Level (CWL) for back analysis to decide design shear strength.

For your reference, normally in Japan, HWL and CWL are decided from observation results of ground water level monitoring, which is not available for this recommendation.

3.4 Estimating Design Shear Strength Parameters

The purpose of design is for measures to stabilize / settle gravity deformation. Thus a supposed slip surface should run within rock mass showing non-circular shape. The proper analysis method is Janbu Method, therefore.

Unit weights were set refereeing to the average value of laboratory rock tests located on the targeted survey lines.

In gravity deformation, bed rock creep is a dominate shear mechanism. In back analysis, therefore, a probable internal friction angle of shear strength is calculated with the condition of cohesion as zero and factor of safety as Fs=1.0.

The slice reading for analysis are shown in Appendix-5: Slice Reading.

3.5 Examining Groundwater Drainage (Horizontal Drainage Borings)

In the design of countermeasures against landslides, the effects by horizontal drainage boring are considered as a certain meters decrease of groundwater level in slope stability analysis, which positively affects a factor of safety.

In this recommendation, the effects by horizontal drainage boring are considered as a 5 meters decrease of groundwater level, due to the following reasons.

In Japan, the effects by horizontal drainage borings from the surface are considered as a 3 meters decrease of groundwater level, whereas those by a drainage well, which installs horizontal drainage borings from subsurface, are considered as a 5 meter decrease. The reason why the effects by a drainage well are considered larger is that the horizontal drainage borings from a drainage well can be placed along the targeted slip surface of a landslide with clay. Thus effective drainage sections of drainage borings from a drainage well are longer than those of drainage boring from the surface as show in the figure on the left below.

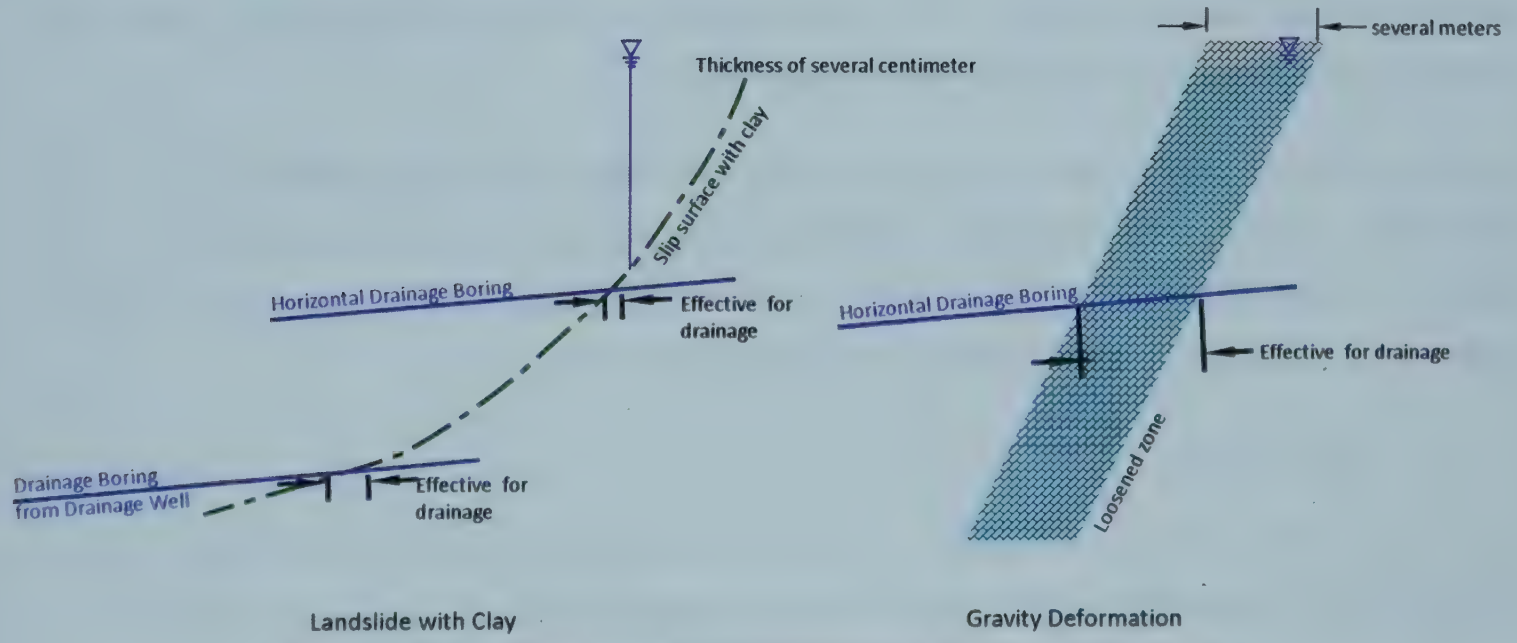


Figure 3-1 Difference in Efficiency of Drainage Borings

JICA Project Team

Unlike slip surfaces of landslide with clay, loosened zones of gravity deformation or bedrock creep have a certain thickness of several meters. Thus effective drainage sections are considered much longer than those for landslide with clay, in which a thickness of slip surface is usually several centimeter. As shown in the figure above in the right, even from the surface, effects by horizontal drainage borings are expected to be equal or more than those from a drainage well. Thus the effects by horizontal drainage borings to stabilize gravity deformation are considered as a 5 meters decrease of groundwater level.

In Japan, for design of countermeasures against landslide with clay, the maximum increase of factor of safety by groundwater control works including horizontal drainage boring is limited as 5 % or less. Considering the aforementioned much higher efficiency of horizontal drainage borings to stabilize gravity deformation as shown in the figure above, in this recommendation, no limitation for the maximum increase of factor of safety was set.

In Appendix-6: Image of Control Works, arrangements of horizontal drainage borings are shown along with earth removal works described below.

3.6 Examining Earth Removal Works

To stabilize a gravity deformation block, direct weight removal should work effectively. On the other hand, an internal friction angle of shear strength of a loosened zone should be larger and difference with an inclination of slip surface should be smaller. Thus effective locations for weight reduction is limited to a narrow section just under a head scarp. Considering the aforementioned limitation, effectiveness and feasibility of earth removal works were examined. The arrangements of the works with the maximum possible soil removal are recommended in Appendix-6.

Since the risk of gravity deformation was not observed as numerical data by landslide monitoring, in this recommendation, moderate design factor of safety was adopted as DFs=1.1 excluding Line-3. On Line-3, gravity deformation is significantly observed by visual inspection, the design factor of safety of DFs=1.15 was adopted.

4 Conclusion

In this recommendation, instability of gravity deformation at the site of NH717 and probable risk of deep-seated landslide caused by neglecting instable blocks are focused.

By examining the results of core boring and groundwater variation analysis chart, loosened zones of gravity deformation or bedrock creep were confirmed. This supports the idea that the site is under possible threats of gravity deformation. And based on these examination, arrangements of control measures are recommended.

Other than the countermeasures, magnitude of the risk of gravity deformation and the actual deformation should be confirmed through surface / subsurface displacement monitoring. Depending on the risk confirmed through displacement monitoring, a design factor of safety higher than those used in this recommendation may be adopted.

Details of soil removal works on plan view should be further considered by the local

consultant based on site survey with detailed cross sections.

As for groundwater drainage works, the followings should be noted.

- Large groundwater drainage works may decrease groundwater level for the wells used by local residents. Thus compensation should be considered.
- Groundwater monitoring should be continued to verify the effects of the works.
- The recommended groundwater drainage works are supposed to ease the pressure of groundwater. Thus the effects are expected only in the limited period in rainy seasons. Constant dewatering may not be expected.
- Periodic maintenance by cleaning the dewatering pipes are recommended.

Appendix 1

Requirements on TOR

APPENDIX-I

TERMS OF REFERENCE (TOR)

(To be Applicable as Relevant to the Assignment)

Consultancy Services for preparation of Detailed Project Report for Slope Protection work of section of Km. 25.60 to Km 26.1 (Bagrakot -Kafer) of NH-717A in the state of West Bengal **Terms of Reference for Consultancy Services (TOR)**

1. General

1.1 The National Highways and Infrastructure Development Corporation (NHIDCL)has been entrusted with the assignment of "preparation of Detailed Project Report for Slope Protection work of section of Km. 25.60 to Km 26.1 (Bagrakot -Kafer) of NH-717A in the state of West Bengal ". NHIDCL now invites proposal from Technical consultants for carrying out detailed project report as per details given in **Annexure-1.**

1.2 NHIDCL will be the employer and executing agency for the consultancy services and the standards of output required from the appointed consultants are of international level both in terms of quality and adherence to the agreed time schedule. The consultancy firm will solely be responsible for submission of quality work in stipulated period.

1.3 Deleted.

1.4 Background

1.4.1 For expert advice the matter was referred to the Japan International Cooperation Agency (JICA). Their experts carried out site visits and their site visit reports are enclosed as Appendix-XI for reference and incorporation of suitable measures to take care of the specific issues brought out in their site report in this RFP. The JICA experts also provide Draft Guidelines regarding planning, slope stability, bridge and tunnel are enclosed for perusal and incorporation of suitable measures in this RFP (Appendix-XII). To understand gravity deformation, the draft slope stability guideline should be well studied. In addition the JICA expert also provides TOR for Topographical Survey and Drilling as well as requirements of surveys, including topographic mapping, drilling survey, geophysical exploration, and landslide monitoring, and the same have also been part of this RFP.

1.5 Specific Requirement of surveys in compliance to JICA directions:

Contents of Surveys

The survey shall aim to provide the following information:

1) To prepare detailed topographic map with contour lines,
2) To map geological conditions relating to the landslide and blocks of gravity deformation, and
3) To evaluate extent and stability of the blocks of gravity deformation.

The requirements of the survey are explained in Table 0-1 ~ Table 0-6. The survey plan is shown in Figure 0-1.

Table 0-1 Requirements of Detailed Topographic Survey

Item	Specification	Purpose
Topographic Survey **Plan Survey:** 0.8km² **Profile Survey:** **Line-1:** 860m **Line-2:** 700m **Line-3:** 930m **Line-4:** 320m **Line-5:** 520m Center line of NH717 shall be shown in the map and profile. The origin of the distance on the profile shall be the center line of NH717.	• Plan survey shall prepare a topographic contour map with symbols of open cracks, step-formed displacement, scarps, cliffs, and cracks with level differences. • The areas to be mapped is shown in Figure 0-1, which is same as recommended in Ref. 1. The mapping area shall include the ridge lines and the blocks of gravity deformation colored in Figure 0-1 as well as the premises and buildings of the school located on the ridge. • Scale of the map shall be 1/2,000, 1/2,500 or 1/3000. • Intervals of contour lines shall be 2 meters in elevation. The contour lines of every 10 meters shall be drawn with bold line as indices. • Control points shall be placed on the ground within the mapped area. • On the line of profiles, short wooden stakes shall be placed on the ground at every 50 meters distance from the center line of NH717 as well as at each point of change of terrain, such as changes in gradient of slope (knick lines) and shoulder of displacement. • Control and reference points, where plan and profiles are correlated, shall be prepared for each profile and stakes shall be placed on the ground. • In order to ensure the accuracy of profile survey, minimum clearing of vegetation shall be performed as required to secure visibility of a total station.	• Identification of blocks of gravity deformation • Base map for geological field survey • Base map for design of counter measures (plan and profile). • Reference points on site for field surveys • Preparation of 3D Cad Data of Terrain Conditions to create profiles for possible options.

Table 0-2 Requirments of Geological Filed Survey

Item	Specification	Purpose
Geological Field Survey: 0.8 km²	• Topographical / geomorphological features and conditions shall be mapped. • Geological conditions shall be mapped, such as, rock/soil types, hardness of rock, weathering conditions of rock, direction of bedding and foliation, and crack conditions (crack interval, aperture and direction of cracks, and conditions of cracks). • Abnormality suggesting landslide and gravity deformation shall be mapped, such as topographic features of landslide/ gravity deformation as shown in Chapter 1.1.3, 1.1.4, 1.2.3, and 1.3.3 of the Draft Slope Guideline in Ref. 7.	• Identification of blocks of gravity deformation • Essential information for design of counter measures (plan and profile).

Table 0-3 Requirments of Drilling Survey

Item	Specification	Purpose
Drilling / boring survey **BV-1:** 60m (W, D) **BV-2:** 60m (W, D) **BV-3:** 40m (W) **BV-4:** 40m (W, D) **BV-5:** 40m (W, D) **BV-6:** 40m (W, D)	• Borehole diameter shall be 66 mm or larger. • Drilling / boring survey shall be performed in accordance with the recommendation as per ToR • Drilling shall obtain core samples with core recovery ratio of 95% or more and use drilling water with/without additives to avoid burning of core samples by the heat of drilling. • All core samples obtained through drilling shall	• Understanding soil / rock properties. • Interpreting geological structure • Understanding depth and degree of loosened zone of bedrock →Assuming shape and

BV-7:	40m (W)
BV-8:	40m (W, D)
BV-9:	40m (W, D)

Borehole Treatment after Drilling

W: Groundwater monitoring well

D: Underground displacement monitoring

be stored in core boxes, which can contain core samples of 1 meter length in a slot. Samples shall be stored in the original order and directions. Core boxes shall be lined with plastic sheet to avoid rapid drying. The sections where core samples were lost shall be separated with partition and kept blank.

- Photos of core samples shall be captured and organized in accordance with the recommendation in Ref. 4.
- Geological conditions shall be properly organized into drilling logs in accordance with the recommendation in Ref. 4.
- Daily report of drilling shall be organized in accordance with the recommendation in Ref. 4, especially depth of water inflow and loss, water table before and after the work, depth of casing installation, drilling speed, flow volume and pressure of drilling water, loading to drilling bit, and rotation speed shall be recorded.
- After reaching the planned depth, Perforated PVC pipes shall be installed into each borehole as groundwater monitoring well. PVC pipes shall have internal diameter of 40 mm or more and perforated sections shall be from 1.0 meter from the ground to the bottom of the borehole and protected by thin geotextile filter.
- Groundwater monitoring well shall have 1.0 meter high standing pipe from the ground with basement of concrete.
- For boreholes with "D" on the left column, bore hole extensometer shall be installed in accordance with the recommendation in Ref. 6. The borehole extensometer shall have three cables with sheaths to be fixed at different depth, which are to cover the depth of slip surface or loosened zone.
- Direct observation of borehole wall shall be performed with borehole cameras or borehole imagery equipment, before installing casing pipes right after drilling of certain section. It is highly likely that drilling water will be lost while drilling proceeds.

depth of slip surface for design.
→Designing effective earth removal
→Setting required resistance force

- Understanding groundwater condition
  →Identifying groundwater storage to be drained by control work
- Evaluating stability of landslide and gravity deformation
  →Prioritizing landslide and gravity deformation blocks as well as orders of countermeasure construction.

Table 0-4 Resistivity Survey

Item	Specification	Purpose
Electrical Resistivity Survey Line-1: 860m Line-2: 700m Line-3: 930m Line-4: 320m Line-5: 520m **Shall be conducted in both rainy and dry seasons.**	• Survey lines shall be prepared to secure a depth of exploration up to 100 meters or more. • Resistivity distribution maps shall be prepared at specified elevations. • Resistivity profile shall be prepared for each survey line. • Changing rate of resistivity shall be shown in the profiles of each survey line. • Difference analysis with results from different periods shall be performed. Survey shall be conducted once in both rainy and dry seasons.	• Understanding depth and degree of loosened zone of bedrock →Assuming shape and depth of slip surface for design. →Designing effective earth removal →Setting required resistance force • Understanding groundwater condition →Identifying groundwater storage to be drained by control work

Table 0-5 Seismic Exploration

Item	Specification	Purpose
Refraction Seismic Survey Line-3: 930m	• Survey lines shall be prepared to secure a depth of exploration up to 10 meters or more. • Seismic speed profile shall be prepared for each survey line.	• Estimating thickness of debris and depth of bedrock →Assuming shape and depth of slip surface for design →Designing effective countermeasures.

Table 0-6 Landslide Monitoring

Item	Specification	Purpose
Monitoring	[Rainfall observation] • Rain gauge for consecutive observation shall be installed at the site. In case rain gauge for consecutive observation is difficult to procure, daily measurement of rainfall by locals, such as by the school, may substitute. • Daily precipitation data of the nearest meteorological stations shall be collected and analyzed.	• Improving accuracy of the factor of safety of the current condition. • To evaluate effectiveness of groundwater drainage borings in future. • Grasping critical conditions of gravity deformation • To evaluate effectiveness of countermeasures in future. • To formulate thresholds for an early warning system in future.
	[Groundwater monitoring] • Groundwater level in the monitoring wells shall be observed every week and summarized as groundwater level fluctuation charts.	• Evaluating stability of landslide and gravity deformation →Prioritizing landslide and gravity deformation blocks as well as orders of countermeasure construction.
	[Monitoring of underground displacement] • The displacement of the borehole extensometers shall be observed every month and summarized as an extensometer displacement charts.	

(Fig-1)Topographic Survey Area

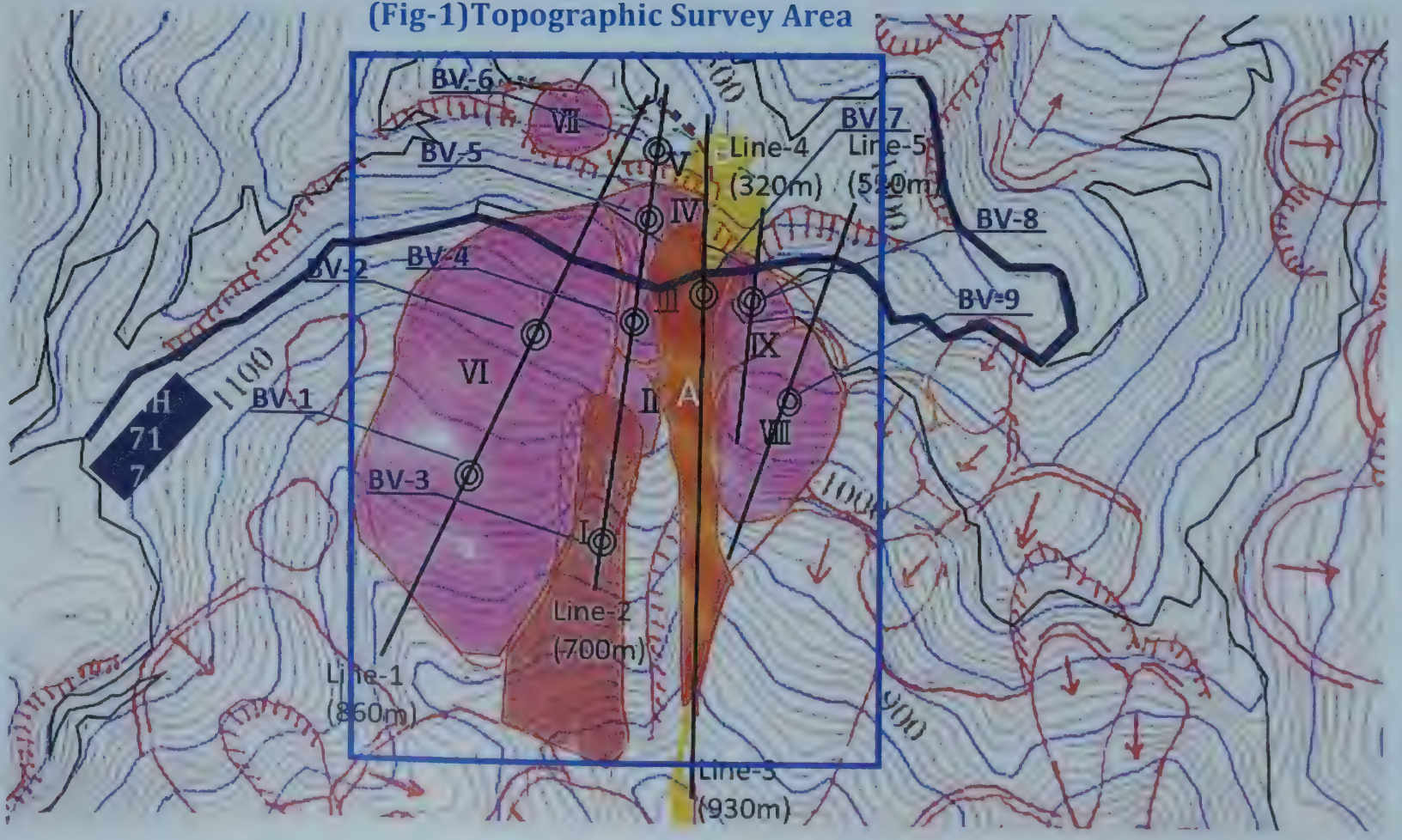


Figure 0-1 Area to Be Mapped and Survey Plan

(Supplimental Information)

Priority of survey

1st Priority:
All topographic survey
Geological field survey
Drilling survey at all boreholes excluding BV-7
Electrical resistivity survey of all survey lines excluding Line 3
Rain fall observation.
Monitoring of groundwater and underground displacement at all boreholes excluding BV-7

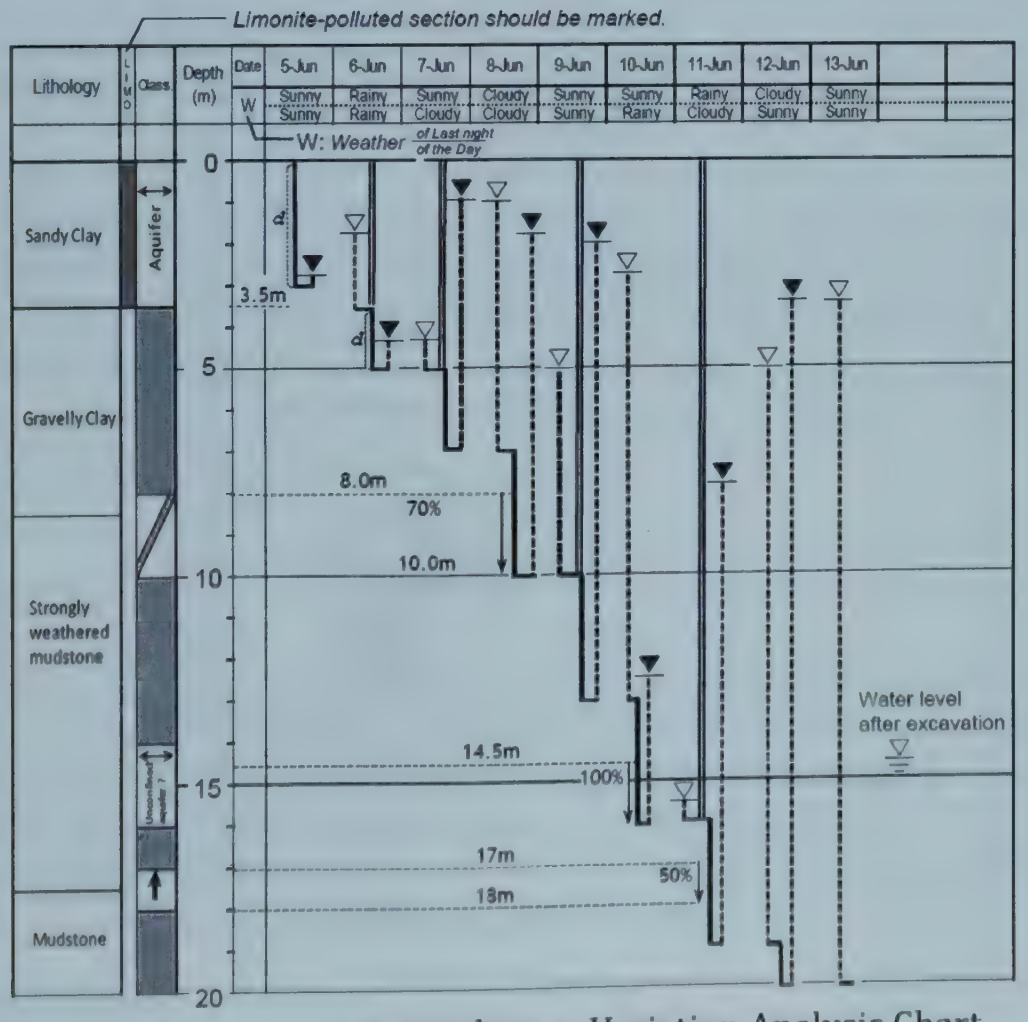
2nd Priority:
Drilling survey at BV-7
Electrical resistivity survey of Line 3
Refraction seismic survey of Line 3
Monitoring of groundwater and underground displacement at BV-7

Borehole cameras or Borehole imagery

Direct observation of borehole wall shall be planned if the Consultant are not confident to obtain core nples of more than 75 % of sampling ratio. In case the Consultant are confident to do so but the obtained core nples by the Consultant are not satifsfy the core sampling ration of 75, the Consultant shall execute boring with ehole cameras or borehole imagery.

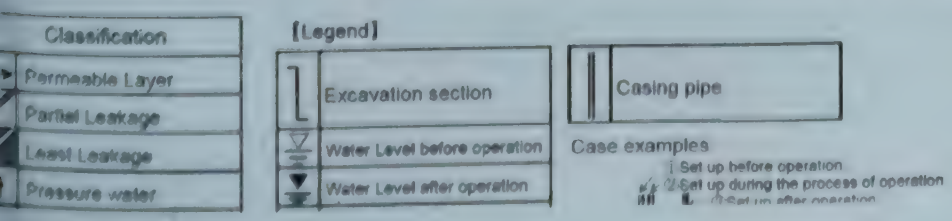
Mesurement of watertable in boreholes before and after the drilling work of each y.

Measurement of groundwater tabel in boreholes being drilled is crutially important for design of ntermeasures against gravity deformation. Thus measurement of groundwater leven in boreholes every day ore and after the drilling work is essential and shall be properly recorded in daily drilling record shown in pendix-IX and X and organized into groundwater variation analysis chart.



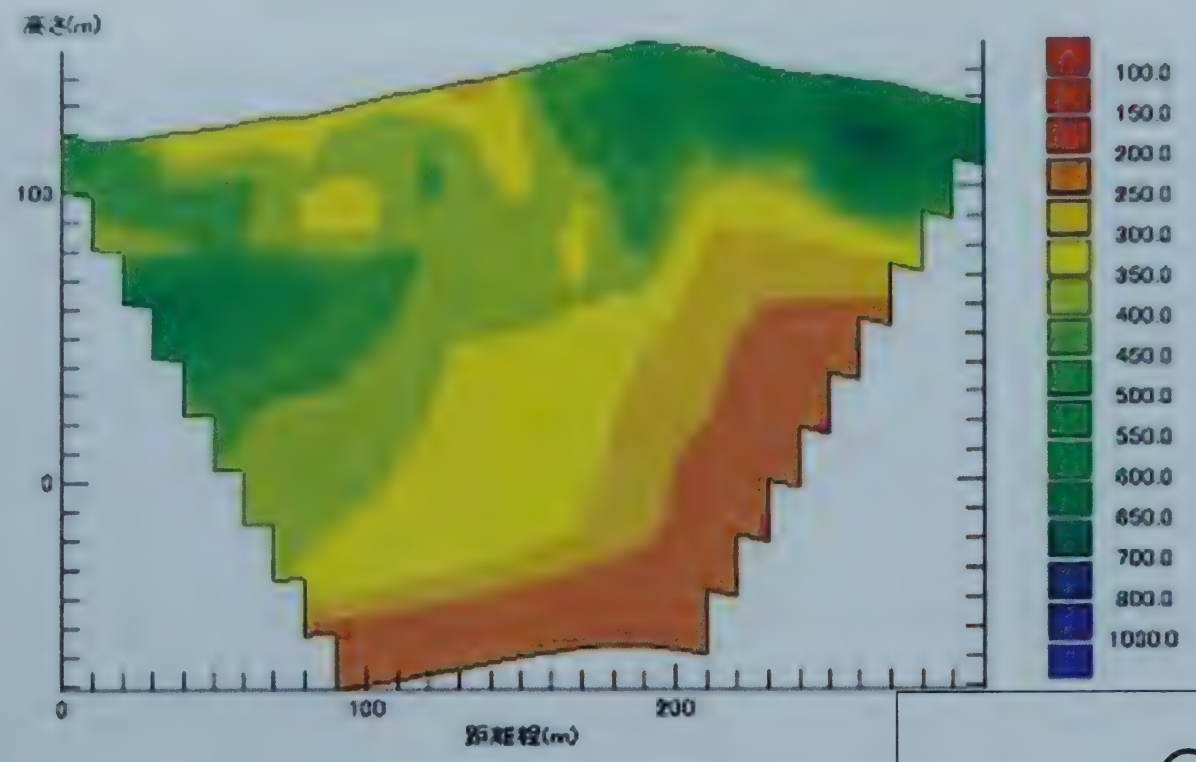
Groundwater Variation Analysis Chart

Source: Shin, Y.S., Jisuberi Kogaku (Landslide Engineering) -Theory and Practice-, Sankaido, 1989.



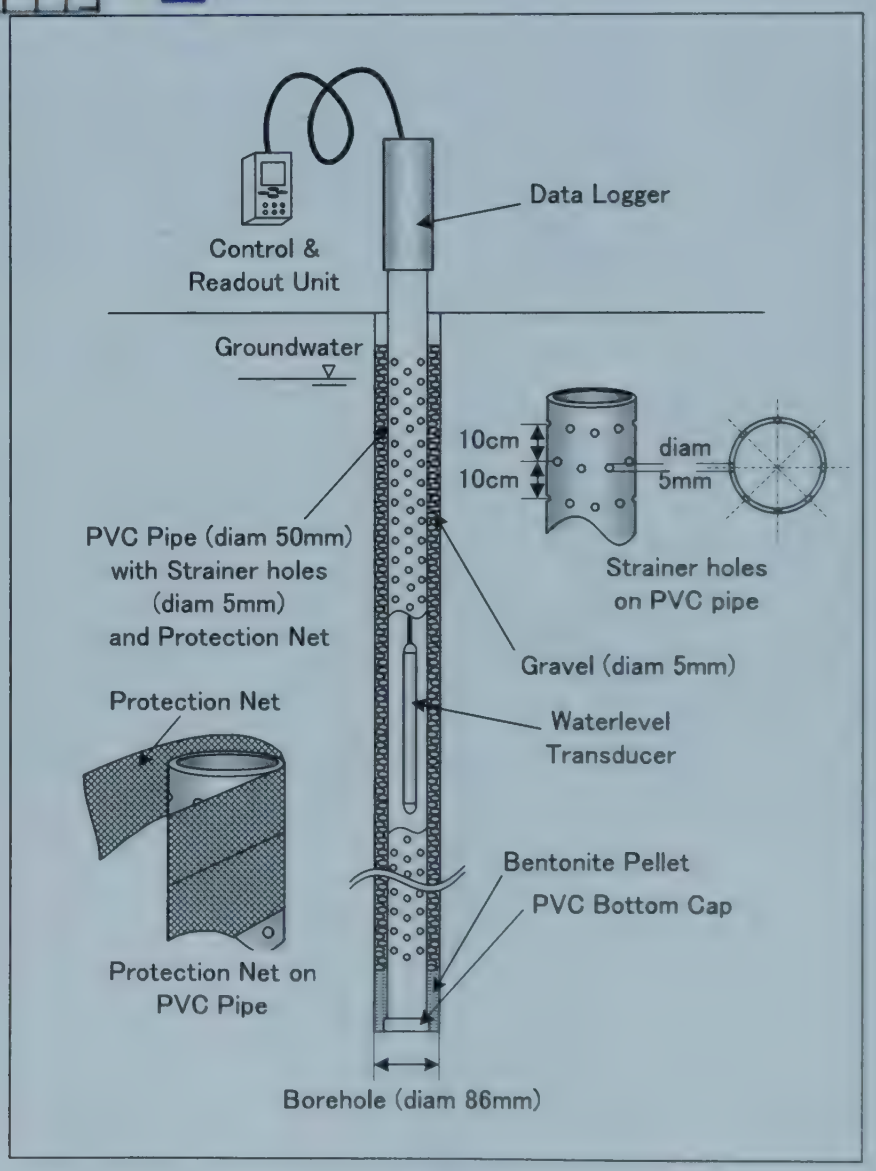
Electrical Resistivity Survey

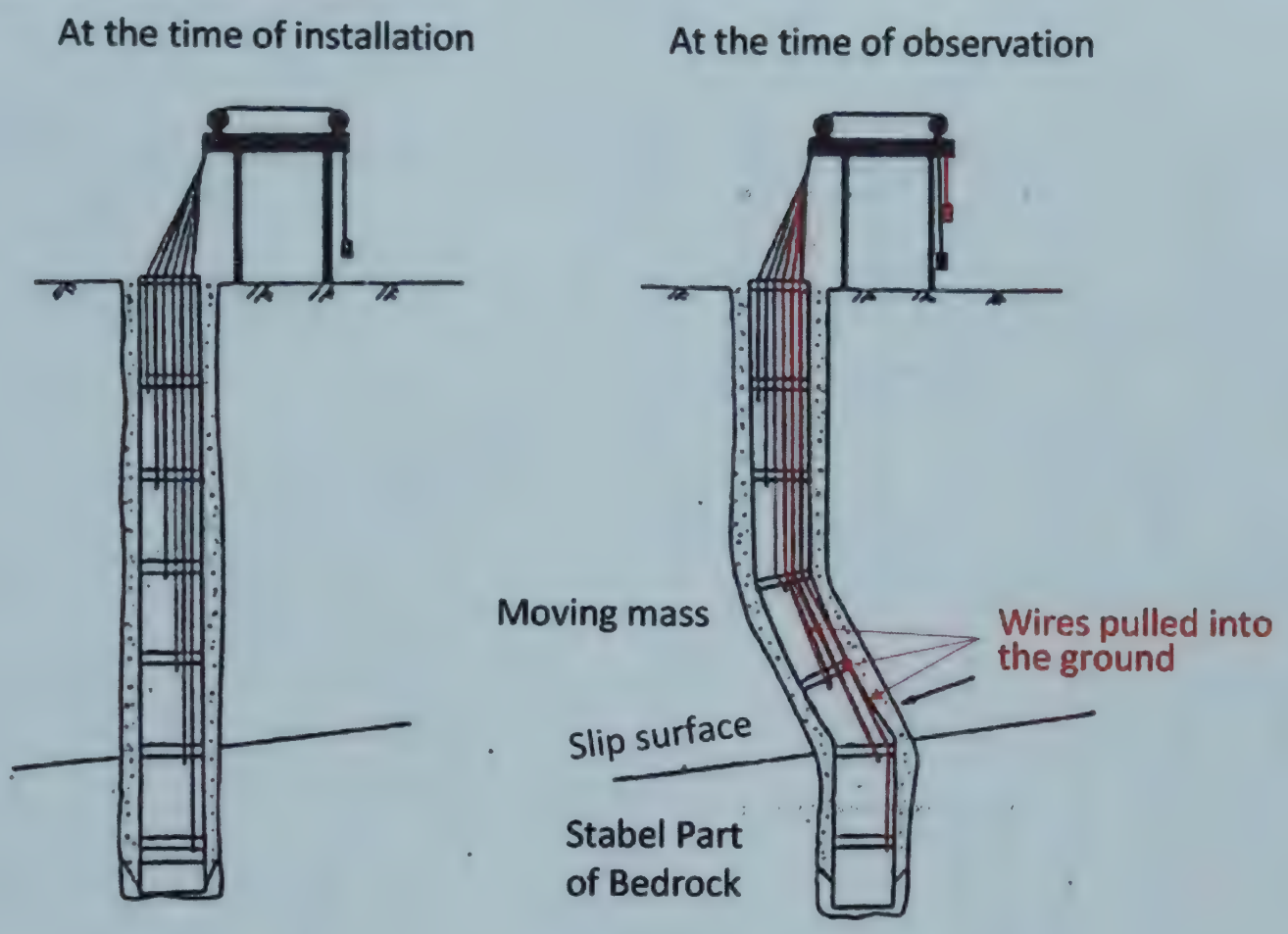
The electrical resistivity survey should be 2 dimention along the survey lines and the results should b organized by geo-tomography method, such as shown below:



Groundwater Monitoring Well

Typical image of groundwater monitoring is on the left.





Borehole Extensometer

Borehle extensometer is a simple and effective monitoring method for underground displacement.

At least three wires shall be attached to the pipe of groundwater monitoring well. The wires shall be fixed at different depths on the pipes, which shall cover the depth of possible slip surface.

During installation of the pipes into a borehole, the wires shall be fixed lightly to the pipes with vynal tapes so that the wires can keep proper tension to detect any deformation underground. Another end of the wires shall be supported by an observation stand on the ground with appropirate weights to keep tension of wires. The observation stand shall have scales/measures to observe displacements from the previous observation, with some markings on the wires.

Survey Period

The surveys recommended in the previous chapter shall be carried out in the coming dry season and the results shall be submitted before the end of March 2020, excluding monitoring and electric resistivity survey in rainy season. Monitoring shall start in the dry season and cover the rainy season in 2020. The electric resistivity survey in rainy season shall be conducted in the rainy season in 2020. The result of monitoring and electric resistivity survey in rainy season shall be submitted by the 15th of September 2020.

2. Objective

The survey shall aim to provide the following information:

1) To prepare detailed topographic map with contour lines,
2) To map geological conditions relating to the landslide and blocks of gravity deformation, and
3) To evaluate extent and stability of the blocks of gravity deformation.

Appendix 2

Plan View for Survey

Target block 4

Target block 5

Target block 3

Target block 2

Target block 1

BV-3

BV-8 Recommended BV-8A

BV-7

Recommended BV-7A

Line-4 (520m)

Line-5 (520m)

Line-3 (930m)

projected BV-3

projected Line-2 (930m)

BV-6

projected BV-6

BV-5

projected BV-5

projected BV-4

BV-4

BV-3

Line-2 (700m)

BV-2

BV-1

Line-1 (860m)

500m

0

04-21

Appendix 3

Borehole Record Analysis

ly Drilling Operation Report

ect Name: Landslide Protection Project

hole Number: ..... BV 1 Line-1

: 21/4/2021

tion: ..... Barbot ,Pkg. IV-C NH717A

IME :mm)	Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
To	hh:mm	minute	From	To	Meter drilled								From [Gl-m]	To [Gl-m]	Total [m]	Number of times
7:45	0:45	45	0	0.5	0.5	1.11111111	nil	weathered Rock	grey	15	12	20				
8:45	1:00	60	0.5	1.5	1	1.66666667	nil	weathered Rock	grey	18	13.5	25	1.5	1.6	0.1	90
9:45	1:00	60	1.5	3	1.5	2.5	0.23	weathered Rock	brown	20.5	10.5	48.78049				
10:45	1:00	60	3	4.5	1.5	2.5	0.26	weathered Rock	brown	21	13.5	35.71429				
11:15	0:30	30	4.5	6	1.5	5	0.2	weathered Rock	brown	22.5	12.5	44.44444				
12:15	1:00	60	6	7.5	1.5	2.5	0.26	weathered Rock	brown	24	11.9	50.41667				
13:15	1:00	60	7.5	9	1.5	2.5	nil	weathe		.5	14.3	33.48837	9	9.1	0.1	80
14:15	1:00	60	9	10.5	1.5	2.5	nil	weathe		0.5	13.3	35.12195	10.5	10.6	0.1	80
8:30	1:30	90	10.5	12	1.5	1.66666667	nil	weathe		24	13.5	43.75	12	12.05	0.05	80
10:30	2:00	120	12	13.5	1.5	1.25	nil	weather	Black	22.5	12.5	44.44444	13.5	13.55	0.05	60
12:00	1:30	90	13.5	15	1.5	1.66666667	0.12	wea ed Rock	Black	23	14.2	38.26087				
13:30	1:30	90	15	16.5	1.5	1.66666667	0.12	athered Rock	Black	20.5	11.9	41.95122				
9:45	1:45	105	16.5	18	1.5	1.42857143	.3	hard rock	Black	21.4	12.5	41.58879				
12:00	2:00	120	18	19.5	1.5	1.25	0.4	hard rock	Black	22.4	13.6	39.28571				
13:30	1:30	90	19.5	21	1.5	1.66666667	0.4	hard rock	Black	23.2	14.2	38.7931	21	21.05	0.05	60
17:00	1:30	90	21	22.5	1.5	1.66666667	0.27	hard rock	Black	24.2	13.9	42.56198	22.5	22.6	0.1	80
10:00	2:00	120	22.5	24	1.5	1.25	0.33	Hard Rock	Black	24	18.1	24.58333				
11:30	1:30	90	24	25.5	1.5	1.66666667	0.4	Hard Rock	Black	21.1	12.6	40.28436				
13:30	2:00	120	25.5	27	1.5	1.25	0.43	Hard Rock	Black	21.1	12.7	39.81043				
16:00	1:30	90	27	28.5	1.5	1.66666667	0.4	Hard Rock	Black	24	18.3	23.75				
17:00	1:00	60	28.5	30	1.5	2.5	0.41	Hard Rock	Black	21.9	18.9	13.69863				
18:30	1:30	90	30	31.5	1.5	1.66666667	0.41	Hard Rock	Black	21.9	18.9	13.69863				
8:30	1:00	60	31.5	33	1.5	2.5	nil	weathered Rock	Black	24.4	14.9	38.93443	33		0.05	50
10:30	2:00	120	33	34.5	1.5	1.25	nil	weathered Rock	Black	23.5	16.5	29.78723	34	34.55	0.05	60
12:30	2:00	120	34.5	36	1.5	1.25	nil	weathered Rock	Black	21.9	15.4	29.68037	36	36.05	0.05	60
17:00	2:00	120	36	37.5	1.5	1.25	nil	weathered Rock	Black	22.6	15.9	29.64602	37.5	37.55	0.05	60
9:00	2:00	120	37.5	39	1.5	1.25	nil	weathered Rock	BROWN	23.5	16.5	29.78723	39	39.05	0.05	50
11:00	2:00	120	39	40.5	1.5	1.25	nil	weathered Rock	BROWN	22	15.9	27.72727	40.5	40.55	0.05	56
13:00	2:00	120	40.5	42	1.5	1.25	nil	weathered Rock	BROWN	20.9	14.6	30.14354	42	42.08	0.08	69
17:00	2:00	120	42	43.5	1.5	1.25	nil	weathered Rock	BROWN	24.3	15.6	35.80247	43.5	43.56	0.06	60
9:00	2:00	120	43.5	45	1.5	1.25	nil	weathered Rock	BROWN	21.9	16.5	24.65753	45	45.05	0.05	55
10:00	1:00	60	45	46.5	1.5	2.5	nil	weathered Rock	BROWN	23.3	14.6	37.33906	46.5	46.55	0.05	60
12:00	2:00	120	46.5	48	1.5	1.25	nil	weathered Rock	BROWN	22.8	13.9	39.03509	48	48.1	0.1	80
14:00	2:00	120	48	49.5	1.5	1.25	nil	weathered Rock	BROWN	24.3	15.6	35.80247	49.5	49.59	0.09	77
9:00	2:00	120	49.5	51	1.5	1.25	nil	weathered Rock	BROWN	24.5	16.5	32.65306	51	51.05	0.05	50
11:00	2:00	120	51	52.5	1.5	1.25	nil	weathered Rock	BROWN	22.9	15.6	31.87773	52.5	52.56	0.06	64
13:30	2:30	150	52.5	54	1.5	1	nil	weathered Rock	BROWN	21.9	14.7	32.87671	54	54.07	0.07	70
8:00	2:00	120	54	55.5	1.5	1.25	nil	weathered Rock	BROWN	23	15.6	32.17391	55.5	55.56	0.06	68
10:30	2:30	150	55.5	57	1.5	1	nil	weathered Rock	BROWN	22.6	14.5	35.84071	57	57.05	0.05	55
10:00	2:00	120	57	58.5	1.5	1.25	nil	eathered Rock	BROWN	22.6	13.9	38.49558	58.5	58.55	0.05	58
12:30	2:30	150	58.5	60	1.5	1	nil	hered Rock	BROWN	24.2	15.6	35.53719	60	60.1	0.1	90
						1.6595045						34.3226				

Hard and soft

Strong leakage zone

Low leakage zone

Stable ground water

Hard rock

nterpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-31.5m or deeper. On the other hand, drilling speed slowed down at a depth of GL-33.0m or deeper, which suggests emergence of hard rock at the depth. In addition, leakage ratio decreased at the depth.

Thus the loosened zone of the main deformation is assumed between GL-31.5m and 33.0m and the depth of GL-33.0m is considered as the bottom of the loozened zone.

Below the depth of GL-33.0 m, no or little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. This means no leakage was made below the depth. Thus GL-12.6m was set as the stable water level of non-pressured groundwater.

Appendix

ily Drilling Operation Report

oject Name: Landslide Protection Project

rehole Number: ..... BV 2 Line-1

te: 14/5/2021

cation: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURB SAMPLE)			
m	To	hh:mm	minute	From	To	Meter drilled								From [Gl-m]	To [Gl-m]	Total [m]	Number times
00	8:00	1:00	60	0	1.5	1.5	2.5	nil	weathered Rock	Black	18	12.6	30	1.5	1.6	0.1	
00	9:00	1:00	60	1.5	3	1.5	2.5	nil	weathered Rock	Black	20	13.1	34.5	3	3.05	0.05	
30	10:30	1:00	60	3	4.5	1.5	2.5	nil	weathered Rock	Black	20	16.3	18.5	4.5	4.6	0.1	
30	11:30	1:00	60	4.5	6	1.5	2.5	nil	weathered Rock	Black	18.3	11.9	34.97268	6	6.05	0.05	
30	12:30	1:00	60	6	7.5	1.5	2.5	nil	weathered Rock	Black	20.1	18.5	7.960199	7.5	7.55	0.05	
30	13:30	1:00	60	7.5	9	1.5	2.5	nil	weathered Rock	Black	18.9	11.3	40.21164	9	9.05	0.05	
00	14:40	0:40	40	9	10.5	1.5	3.75	nil	weathered Rock	black	22.1	18.1	18.09955	10.5	10.55	0.05	
40	15:40	1:00	60	10.5	12	1.5	2.5	nil	weathered Rock	black	21.5	18.1	15.81395	12	12.1	0.1	
40	17:10	1:30	90	12	13.5	1.5	1.66666667	0.15	weathered Rock	black	22.2	19.6	11.71171				
00	8:00	1:00	60	13.5	15	1.5	2.5	0.25	hard rock	Black	21.3	18.6	12.67606				
00	10:00	1:00	60	15	16.5	1.5	2.5	0.3	weath		22.2	18.1	18.46847				
00	11:00	1:00	60	16.5	18	1.5	2.5	nil	weath		21.4	16.3	23.83178	18			
00	12:30	1:30	90	18	19.5	1.5	1.66666667	nil	weath		19.3	16.1	16.58031	19.5			
30	14:30	2:00	120	19.5	21	1.5	1.25	nil	weathe	Black	18.9	11.4	39.68254	21		0.1	
30	15:30	1:00	60	21	22.5	1.5	2.5	nil	eathered Rock	Black	24	18.9	21.25	22.5	55	0.05	
30	17:30	2:00	120	22.5	24	1.5	1.25	nil	weathered Rock	Black	24	19.6	18.33333		24.05	0.05	
00	9:00	1:00	60	24	25.5	1.5	2.5	0.3	hard rock	Black	24	19.3	19.58333				
00	14:00	2:00	120	25.5	27	1.5	1.25	0.4	hard rock	Black	21.4	19.5	8.878505				
00	15:00	1:00	60	27	28.5	1.5	2.5	0.27	hard rock	Black	22.1	19.5	11.76471				
00	16:00	1:00	60	28.5	30	1.5	2.5	0.25	hard rock	Black	22.6	19.5	13.71681				
00	17:10	1:10	70	30	31.5	1.5	2.14285714	0.25	hard rock	Black	19.3	12.1	37.3057				
00	8:00	1:00	60	31.5	33	1.5	2.5	0.33	Hard Rock	Black	24	18.1	24.58333				
00	9:00	1:00	60	33	34.5	1.5	2.5	0.4	Hard Rock	Black	21.1	12.6	40.28436				
00	10:00	1:00	60	34.5	36	1.5	2.5	0.43	Hard Rock	Black	21.1	12.7	39.81043				
00	11:00	1:00	60	36	37.5	1.5	2.5	0.4	Hard Rock	Black	24	18.3	23.75				
00	12:30	1:30	90	37.5	39	1.5	1.66666667	0.41	Hard Rock	Black	21.9	18.9	13.69863				
30	13:40	1:10	70	39	40.5	1.5	2.14285714	nil	weathered Rock	Black	21.3	18.1	15.02347	40.5	40.55	0.05	
40	15:00	1:20	80	40.5	42	1.5	1.875	nil	weathered Rock	Black	23.1	19.3	16.45022	42	42.05	0.05	
00	16:15	1:15	75	42	43.5	1.5	2	nil	weathered Rock	Black	21.6	17.3	19.90741	43.5	43.6	0.1	
15	17:40	1:25	85	43.5	45	1.5	1.76470588	nil	weathered Rock	black	24	11.9	50.41667				
00	10:00	1:00	60	45	46.5	1.5	2.5	nil	weathered Rock	Black	22.1	16.3	26.24434				
00	12:30	2:30	150	46.5	48	1.5	1	nil	weathered Rock	Black	24	18.1	24.58333				
30	14:00	1:30	90	48	49.5	1.5	1.66666667	nil	weathered Rock	Black	23.1	19.3	16.45022	49.5			
00	15:50	1:50	110	49.5	51	1.5	1.36363636	nil	weathered Rock	Black	23.1	18.6	19.48052		5		
50	17:30	1:40	100	51	52.5	1.5	1.5	nil	weathered Rock	Black	21.1	19.3	8.530806		52	0.05	
30	14:00	2:30	150	52.5	54	1.5	1	nil	weathered Rock	Black	23.9	18.3	23.43096	54	54.05	0.05	
00	15:40	1:40	100	54	55.5	1.5	1.5	nil	weathered Rock	Black	21.1	12.6	40.28436	55	55.55	0.05	
40	17:00	1:20	80	55.5	57	1.5	1.875	nil	eathered Rock	Black	24	19.3	19.58333	57	57.05	0.05	
00	8:00	1:00	60	57	58.5	1.5	2.5	nil	ered Rock	Black	24	18.1	24.58333	58.5	58.55	0.05	
00	10:10	2:10	130	58.5	60	1.5	1.15384615	nil	we d Rock	Black	23.7	17.6	25.7384	60	60.05	0.05	
							2.0871142						23.1669				

Hard and soft

Low leakage zone

Strong leakage zone

Low leakage zone

Strong leakage zone

Low leakage zone

Stable ground water

Hard rock

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-52.5m or deeper. On the other hand, abnormally high leakage ratio was obserbed between GL-54.0 and 55.5m and this section is assumed as the loosened zone of the main deformation. The depth of GL-55.5m is considered as the bottom of the loozened zone.

Below the depth of GL-55.5m, no or little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. This means no leakage was made below the depth. Thus GL-16.1m was set as the stable water level of non-pressured groundwater.

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 3

Date: 26/5/2021

Location: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
From	To	hh:mm	minute	From	To	Meter drilled								From [Gl-m]	To [Gl-m]	Total [m]	Number of times
7:00	8:00	1:00	60	0	1.5	1.5	2.5	nil	weathered Rock	Black	21.5	18.1	15.81395	1.5	1.6	0.1	80
8:00	9:10	1:10	70	1.5	3	1.5	2.14285714	nil	weathered Rock	Black	19.9	16.3	18.09045	3	3.15	0.05	68
9:10	10:40	1:30	90	3	4.5	1.5	1.66666667	nil	weathered Rock	Black	21.3	19.1	10.32864	4.5	4.6		
0:40	12:20	1:40	100	4.5	6	1.5	1.5	nil	weathered Rock	Black	24	19.4	19.16667	6	6.05	0.1	75
5:10	16:30	1:20	80	6	7.5	1.5	1.875	nil	weathered Rock	Black	23.5	18.9	19.57447	7.5	7.6	0.05	69
6:30	17:50	1:20	80	7.5	9	1.5	1.875	nil	weathered Rock	Black	24	16.3	32.08333	9	9.05	0.05	55
7:00	8:40	1:40	100	9	10.5	1.5	1.5	nil	weathered Rock	Black	21.5	16.9	21.39535	10.5	10.55	0.05	80
8:40	10:20	1:40	100	10.5	12	1.5	1.5	nil	weathered Rock	Black	21.5	16.9	21.39535				80
4:10	15:40	1:30	90	12	13.5	1.5	1.66666667	nil	weathered Rock	Black	22.1	18.7	15.38462				80
5:40	17:20	1:40	100	13.5	15	1.5	1.5	nil	weathered Rock	Black	23.3	16.9	27.46781				57
6:30	8:00	1:30	90	15	16.5	1.5	1.66666667	nil	weathered rock	Black	23.1	17.4	24.67532	16.5	16.6	0.1	80
8:00	9:25	1:25	85	16.5	18	1.5	1.76470588	0.15	hard rock	Black	22.1	13.9	37.10407				
9:25	11:00	1:35	95	18	19.5	1.5	1.57894737	0.2	hard rock	Black	23.9	17.3	27.61506				
4:40	15:50	1:10	70	19.5	21	1.5	2.14285714	0.25	hard rock	Black	22.9	16.5	27.9476				
6:00	17:05	1:05	65	21	22.5	1.5	2.30769231	0.15	hard rock	[illegible]ck	24	12.8	46.66667				
7:10	18:20	1:10	70	22.5	24	1.5	2.14285714	nil	weathered rock	Bla[illegible]	19.9	17.3	13.06533	24	24.05	0.05	70
7:00	10:15	3:15	195	24	25.5	1.5	0.76923077	nil	weathered rock	[illegible]	[illegible]	18.7	13.82488	25.5	25.6	0.1	80
1:00	13:05	2:05	[illegible]	25.5	27	1.5	1.2	nil	weathered rock	[illegible]	[illegible]	19.9	17.08333	27	[illegible]	[illegible]	[illegible]
5:00	17:20	2:20	[illegible]40	27	28.5	1.5	1.07142857	nil	weathered rock	[illegible]	[illegible]	16.7	30.41667	28.5	[illegible]	[illegible]	[illegible]5
7:00	8:40	1:40	100	28.5	30	1.5	1.5	nil	weathered Rock	Black	23.6	17.8	24.57627	30	[illegible]	[illegible]	[illegible]7
[illegible]	[illegible]	[illegible]	[illegible]40	30	31.5	1.5	1.07142857	nil	weathered Rock	Black	21.6	16.3	24.53704	31.5	[illegible]	[illegible]	[illegible]0
[illegible]	[illegible]	[illegible]	[illegible]20	31.5	33	1.5	1.25	nil	weathered Rock	Black	22.8	17.8	21.92982	33	[illegible]	0.05	60
[illegible]	[illegible]	[illegible]	[illegible]0	33	34.5	1.5	1.36363636	nil	[illegible]ered Rock	Black	20.9	15.5	25.83732	34.5	34.55	0.05	52
:00	13:40	2:40	160	36	37.5	1.5	0.76923077	nil	wea[illegible]	[illegible]	20.8	17.1	17.78846	36	36.05	0.05	70
:00	17:15	2:15	135	37.5	39	1.5	0.9375	nil	wea[illegible]	[illegible]	23.9	19.2	19.66527	37.5	37.55	0.05	60
:00	10:05	2:05	125	39	40	1.5	1.11111111	nil	wea[illegible]	[illegible]	22.2	18.1	18.46847				
							1.2	nil	weathered Rock	Black	23.3	16.6	28.75536	40	40.05	0.05	62
							1.53975863						23.0				

Strong leakage zone

Hard rock

Stable ground water

Low leakage

Hard rock

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-15.0m or deeper. On the other hand, drilling speed slowed down at a depth of GL-24.0m or deeper, which suggests emergence of hard rock at the depth. High leakage ratio was observed between GL-13.5m and 22.5m but leakage decreased at a depth of GL-24.0m or deeper. Thus the depth of GL-24.0m is considered as the bottom of the loozened zone.

Below the depth of GL-24.0 m, no or little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. This means no leakage was made below the depth. Thus GL-9.9m was set as the stable water level of non-pressured groundwater.

Appendix

aily Drilling Operation Report

roject Name: Landslide Protection Project

orehole Number: ..... BV 4

ate: 26/01/2021

ocation: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	BOREHOLE SIZE [mm]	CASING TYPE (mm)	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTU SAMPLE)			
rom	To	hh:mm	minute	From	To	Meter drilled										From [Gl-m]	To [Gl-m]	Total [m]	Numb tim
4:30	17:30	3:00	180	0	1.5	1.5	0.83333333	0.1	weathered Rock	white and grey	92	101.6	20	17	15				
9:15	12:40	3:25	205	1.5	4.5	3	1.46341463	8	Soil	white and grey	92	101.6	18	11	38.88889	1.5	1.95	0.45	16-22-
2:10	5:35	3:25	205	4.5	7.5	3	1.46341463	Nil	Soil	white and grey	92	101.6	20	12	40	3	3.1	0.1	
0:45	12:01	1:16	76	7.5	9	1.5	1.97368421	Nil	Soil	white and grey	92	101.6	18	8	55.55556	7.5	7.6	0.1	
7:30	9:00	1:30	90	9	10	1	1.11111111	0.1	Soil	Black and grey	76	76	19	12	36.84211	9	9.05	0.05	
2:30	5:45	3:15	195	10	13.5	3.5	1.79487179	0.43	weathered rock	Black and grey	76		24	11.6	51.66667				
1:35	13:15	1:40	100	13.5	15	1.5	1.5	0.2	weathered rock	white and grey	76					16.5	16.6	0.1	
2:30	5:25	2:55	175	15	17	2	1.14285714	Nil	weathered rock	white and grey	76								
7:55	11:55	4:00	240	17	20	3	1.25	nil	weathered rock	Black and grey	76					19.5	19.6	0.1	
2:30	5:15	2:45	165	20	22	2	1.21212121	Nil	weathered rock	Black and grey	76								
7:35	10:45	3:10	190	22	24	2	1.05263158	nil	weathered rock	white and grey	76					22.5	22.6	0.1	
3:00	12:40	4:40	28	24	25.5	1.5	0.53571429	nil	weathered rock	Black and grey	76					24	24.05	0.05	
3:20	15:25	2:05		25.5	27	1.5	1.2	56	fractured rock	Black and grey	76					28.5	28.55	0.05	
5:25	17:30	2:05	25	27	28.5	1.5	1.2		fractured rock	Black and grey	76								
3:00	11:20	3:20	200	28.5	30	1.5	0.75	nil	fr d rock	Black and grey	76					30	30.05	0.05	
1:20	13:40	2:	140	30	31.5	1.5	1.07142857	0.25											
4				31.5	33	1.5	2	0.23	fra		76								
5				33	34.5	1.5	1.66666667	nil	fra		76								
3				34.5	36	1.5	0.44117647	0.2	fra		76					34.5	34.55	0.05	
5:40	17:15	1:35	95	36	37.5	1.5	1.57894737	0.23	fractured rock	Black and grey	76								
3:00	12:00	4:00	240	37.5	39	1.5	0.625	0.3	fractured rock	Black and grey	76								
2:00	13:40	1:40	100	39	40	1	1	0.16	fractured rock	Black and grey	76								
							1.22119877								39.7				

Stable ground water

Hard rock

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the bedrock in this borehole is considered to lie at a depth of GL-13.5m or deeper. In the section between GL-17.0m and 22.0m, the groundwater levels after the work increased. In the section deeper than GL-22.0m, the bedrock is considered less loozend . In addition, drilling speed slowed down at a depth of GL-20.0m or deeper, which suggests emergence of hard rock at the depth.

Thus the depth of GL-22.0m is considered as the bottom of the loozened zone.

In this bore hole, the groundwater levels after the work are considered being affected by drilling water. But the groundwater levels before the work of the following working day are stable. Thus GL-8.3m was set as the stable water level of non-pressured groundwater.

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 5

Date: ..... #####

Location: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
From	To	hh:mm	minute	From	To	Meter drilled								From [GI-m]	To [GI-m]	Total [m]	Number of times
8:00	9:00	1:00	60	0	0.5	0.5	0.83333333		DS Taken								
9:00	11:40	2:40	160	0.5	1.5	1	0.625	nil	weathered Rock	Brown	24	16	33.33333	1			35
3:00	16:40	3:40	220	1.5	3	1.5	0.68181818	nil	weathered Rock	Brown	24	11	54.16667				45
8:00	9:40	1:40	100	7.5	9	1.5	1.5	0.25	Hard Rock	Brown	22.8	13.6	40.35088				
9:40	14:00	4:20	260	9	10.5	1.5	0.57692308	0.26	Hard Rock	Brown	24	19.5	18.75				
5:00	16:00	1:00	60	10.5	12	1.5	2.5	0.3	Hard Rock	Brown	19.9	16.5	17.08543				
6:00	17:00	1:00	60	12	13.5	1.5	2.5	nil	weathered rock	Brown	21.5	11.6	46.04651				100
8:00	9:00	1:00	60	13.5	15	1.5	2.5	nil	weathered rock	Brown	24	11.9	50.41667	15			100
9:00	10:00	1:00	60	15	16.5	1.5	2.5	0.26	hard rock	Brown	21	12.3	41.42857				
0:00	14:30	4:30	270	16.5	18	1.5	0.55555556	0.17	hard rock	Brown	22.5	12.6	44				
5:30	17:00	1:30	90	18	19.5	1.5	1.66666667	0.2	hard rock	Brown	20.5	19	7.317073				
8:00	10:30	2:30	150	19.5	21	1.5	1	nil	weathered rock	Brown	25.6	11.9	53.51563		21.05	0.05	70
0:30	12:30	2:00	120	21	22.5	1.5	1.25	nil	weathered rock	Brown	21	16.5	21.42857	22.5	22.55	0.05	70
2:30	14:30	2:00	120	22.5	24	1.5	1.25	nil	weathered rock	Brown	20.3	16.5	18.71921	24	24.05	0.05	80
5:30	17:00	1:30	90	24	25.5	1.5	1.66666667	nil	weathered rock	Brown	21.5	16.4	23.72093	25.5	25.55	0.05	70
8:00	10:00	2:00	120	25.5	27	1.5	1.25	nil	weathered rock	Brown	24	16.9	29.58333	27	27.05	0.05	70
0:00	11:30	1:30	90	27	28.5	1.5	1.66666667	nil	weathered rock	Brown	21	19.3	8.095238	28.5	28.55	0.05	70
1:30	13:00	1:30	90	28.5	30	1.5	1.66666667	nil	weathered rock	Brown	21	19	9.52381	30	30.05	0.05	80
3:00	14:00	1:00	60	30	31.5	1.5	2.5	nil	weathered rock	Brown	24	11.9	50.41667	31.5	31.55	0.05	70
5:00	16:00	1:00	60	31.5	33	1.5	2.5	nil	weathered rock	Brown	21	12.3	41.42857	33	33.1	0.1	80
6:00	17:30	1:30	90	33	34.5	1.5	1.66666667	nil	weathered rock	Brown	22.6	15.4	31.85841	34.5	34.05	0.05	80
8:00	9:00	1:00	60	34.5	36	1.5	2.5	0.3	hard rock	Blackish Grey	23	11.9	48.26087				
			210	36	37.5	1.5	0.71428571	0.25	hard rock	Blackish Grey	24	12.6	47.5				
			190	37.5	39	1.5	0.78947368	0.5	hard rock	Blackish Grey	22.1	19.3	12.66968				
6:40	17:10	0:30	30	39	40	1.5	5	0.6	hard rock	Blackish Grey	21.5	16.3	24.18605				
							1.67438892						32.2				

Strong leakage zone

Low leakage zone

Hard rock

Stable ground water

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-19.5m or deeper. In addition, drilling speed slowed down at a depth of GL-16.5m or deeper, which suggests emergence of hard rock at the depth. Leakage ratio decreaced at a depth of 18.0m or deeper. In the section between GL-13.5m and 19.5m, the groundwater levels after the work increased. Thus the depth of GL-19.5m is considered as the bottom of the loozened zone.

In this bore hole, the groundwater levels after the work are considered being affected by drilling water. But the groundwater levels before the work of the following working day are stable. Thus GL-16.5m was set as the stable water level of non-pressured groundwater.

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 6

Date: 2021/12/6

Location: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
From	To	hh:mm	minute	From	To	Meter drilled								From [Gl-m]	To [Gl-m]	Total [m]	Number of times
7:00	8:00	1:00	60	0	1.5	1.5	2.5	nil	weathered Rock	grey	21.5	17.3	19.53488	1.5	1.95	0.45	4
8:00	9:00	1:00	60	1.5	3	1.5	2.5	nil	weathered Rock	grey	22.3	17.3	22.42152	3	3.45	0.45	5
9:00	10:15	1:15	75	3	4.5	1.5	2	nil	weathered Rock	grey	21.5	18.3	14.88372	4.5	4.6	0.1	6
10:15	11:10	0:55	55	4.5	6	1.5	2.72727273	nil	weathered Rock	grey	24.1	12.9	46.47303	6	6.1	0.1	7
11:10	14:00	2:50	170	6	7.5	1.5	0.88235294	nil	weathered Rock	grey	21.1	12.9	38.86256	7.5	7.55	0.05	5
7:00	8:00	1:00	60	7.5	9	1.5	2.5	nil	weathered	grey	22.1	13.3	39.819	9	9.1	0.1	8
8:00	9:00	1:00	60	9	10.5	1.5	2.5	nil	weathered	grey	21.1	12.3	41.70616	10.5	10.6	0.1	7
9:00	10:10	1:10	70	10.5	12	1.5	2.14285714	nil	weathered	grey	24.3	16.9	30.45267	12	12.05	0.05	6
10:10	11:00	0:50	50	12	13.5	1.5	3	nil	weathered	grey	20.9	11.9	43.0622	13.5	13.6	0.1	9
11:00	12:00	1:00	60	13.5	15	1.5	2.5	nil	weathered	grey	20.9	17.3	17.22488	15	15.05	0.05	7
12:00	13:00	1:00	60	15	16.5	1.5	2.5	nil	weathered	grey	20.5	11.3	44.87805	16.5	16.55	0.05	5
13:00	14:00	1:00	60	16.5	18	1.5	2.5	nil	weathered	grey	22.1	19.6	11.31222	18	18.1	0.1	8
14:00	15:00	1:00	60	18	19.5	1.5	2.5	nil	weathered	grey	21.6	12.9	40.27778				7
15:00	16:00	1:00	60	19.5	21	1.5	2.5	nil	weathered	grey	21.1	12.3	41.70616				8
16:00	17:00	1:00	60	21	22.5	1.5	2.5	nil	weathered	grey	21.1	16.9	19.90521				
17:00	18:00	1:00	60	22.5	24	1.5	2.5	nil	weathered	grey	23.3	12.5	46.35193	24			
7:00	8:00	1:00	60	24	25.5	1.5	2.5	0.25	hard rock	grey	21.1	12.3	41.70616				
8:00	9:00	1:00	60	25.5	27	1.5	2.5	0.23	hard rock	grey	21.9	12.6	42.46575				
9:00	10:00	1:00	60	27	28.5	1.5	2.5	0.21	hard rock	grey	22.1	16.9	23.52941				
10:00	11:50	1:50	110	28.5	30	1.5	1.36363636	0.38	hard rock	grey	24.1	12.9	46.47303				
11:50	13:10	1:20	80	30	31.5	1.5	1.875	0.38	hard rock	grey	21.1	16.3	22.74882				
13:10	15:10	2:00	120	31.5	33	1.5	1.25	0.3	hard rock	grey	19.9	16.9	15.07538				
15:10	16:05	0:55	55	33	34.5	1.5	2.72727273	0.33	rock	grey	20.3	12.6	37.93103				
16:05	17:10	1:05	65	34.5	36	1.5	2.30769231	0.3	hard	grey	21.3	18.9	11.26761				
7:00	9:15	2:15	135	36	37.5	1.5	1.11111111	0.28	ha		21.5	16.3	24.18605				
9:15	12:30	3:15	195	37.5	39	1.5	0.76923077	0.27	ha		22.1	19.6	11.31222				
12:30	14:50	2:20		39	40	1	0.71428571	0.24	ha		21.3	17.5	17.84038				
							2.1433597						30.1				

Strong leakage zone

Low leakage zone

Hard rock

Stable ground water

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-24.0m or deeper. The drilling log shows the similar tendency. Drilling speed slowed down at a depth of GL-28.5m or deeper, which suggests emergence of hard rock at the depth. On the other hand, abnormally high leakage ratio was observed between GL-28.5m and 30.0m.

Thus the depth of GL-30.0m is considered as the bottom of the loozened zone.

Below the depth of GL-30.0 m, no or little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. This means no leakage was made below the depth. Thus GL-8.5m was set as the stable water level of non-pressured groundwater.

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 7

Date: 2021/6/6

Location: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
From	To	hh:mm	minute	From	To	Meter drilled								From [Gl-m]	To [Gl-m]	Total [m]	Number of times
7:00	8:00	1:00	60	0	1.5	1.5	2.5	nil	weathered Rock	grey	21.5	16.9	21.39535	1.5	21.1	12.9	1
8:00	9:00	1:00	60	1.5	3	1.5	2.5	nil	weathered Rock	grey	20.5	12.6	38.53659	3	3.45	0.45	51
9:00	10:00	1:00	60	3	4.5	1.5	2.5	nil	weathered Rock	grey	19.3	12.8	33.67876	4.5	4.6	0.1	69
0:00	11:00	1:00	60	4.5	6	1.5	2.5	nil	weathered Rock	grey	24.1	16.3	32.36515	6	6.1	0.1	72
1:00	13:40	2:40	160	6	7.5	1.5	0.9375	nil	weathered Rock	grey	23.3	19.1	18.02575	7.5	7.55	0.05	55
5:40	17:00	1:20	80	7.5	9	1.5	1.875	nil	weathered Rock	grey	24.1	15.9	34.0249	9	9.05	0.05	60
7:00	8:00	1:00	60	9	10.5	1.5	2.5	nil	weathered	grey	24.5	16.9	31.02041	10.5	10.65	0.15	90
8:00	9:00	1:00	60	10.5	12	1.5	2.5	nil	weathered	grey	21.3	17.5	17.84038	12	12.1	0.1	70
9:00	10:15	1:15	75	12	13.5	1.5	2	nil	weathered	grey	22.1	16.3	26.24434	13.5	13.55	0.05	69
9:00	11:40	2:40	160	13.5	15	1.5	0.9375	nil	weathered	grey	21.5	16.1	25.11628	15	15.2	0.2	120
3:10	14:10	1:00	60	15	16.5	1.5	2.5	nil	weathered	grey	21.5	16.1	25.11628	16.5	16.55	0.05	57
4:10	15:10	1:00	60	16.5	18	1.5	2.5	nil	weathered	grey	24.1	18.1	24.89627	18	18.05	0.05	55
5:10	17:20	2:10	130	18	19.5	1.5	1.15384615	nil	weathered	grey	25.3	19.5	22.9249	1			58
7:00	8:00	1:00	60	19.5	21	1.5	2.5	nil	weathered rock	grey	21.5	16.3	24.18605				80
8:00	9:00	1:00	60	21	22.5	1.5	2.5	nil	weathered rock	grey	19.9	14.2	28.64322	2			70
9:00	10:00	1:00	60	22.5	24	1.5	2.5	nil	wea		19.9	12.3	38.19095	24	4.05	0.05	80
0:00	11:00	1:00		24	25.5	1.5	2.5	nil	wea		21.1	16.9	19.90521	25	25.6	0.1	78
1:00	12:00	1:00	60	25.5	27	1.5	2.5	nil	wea		24	18.9	21.25				9
2:00	13:00	1:00	60	27	28.5	1.5	2.5	nil	wea	grey	24.3	19.6	19.34156	28.5			2
3:00	14:00	1:00	60	28.5	30	1.5	2.5		weathered rock	grey	25.1	16.3	35.05976	30			9
4:00	15:30	1:30	110	30	31.5	1.5	1.36363636	0.27	hard rock	gre		9	18.18182				
5:50	17:20	1:30	90	31.5	33	1.5	1.66666667	0.23	hard rock	gre		9	19.91525				
7:00	8:00	1:00	60	33	34.5	1.5	2.5	0.3	hard rock	grey	21.5	19.3	10.23256				
8:00	9:40	1:40	100	34.5	36	1.5	1.5	0.25	hard rock		22.1	20.1	9.049774				
9:40	11:50	2:10	130	36	37.5	1.5	1.15384615	0.2	hard rock	grey	24.1	18.3	24.06639				
1:50	13:40	1:50	110	37.5	39	1.5	1.36363636	0.3	hard rock	grey	21.2	18.3	13.67925				
3:40	16:50	3:10	190	39	40	1	0.52631579	0.25	hard rock	grey	22.5	17.9	20.44444				
							2.01770176						24.2				

Stable ground water

Hard rock

Strong leakage zone

Low leakage zone

Hard rock

Interpretation and basis for design

Based on GVAC shown in Appendix-4, the less loosened zone in this borehole is considered to lie at a depth of GL-19.5m or deeper. Leakage ratio decreaced at the similar depth and deeper. Drilling speed slowed down at a depth of GL-30.0m or deeper, which suggests emergence of hard rock at the depth. The drilling log suggest the samel. On the other hand, abnormally high leakage ratio wa s observed between GL-28.5m and 30.0m.

Thus the depth of GL-30.0m is considered as the bottom of the loozened zone.

Below the depth of GL-30.0 m, no or little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. This means no leakage was made below the depth. Thus GL-8.4m was set as the stable water level of non-pressured groundwater.

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 8

Date: 17/02/2021

Location: ..... Barbot ,Pkg. IV-C NH717A

TIME (hh:mm)		Erapsed time		METERAGE [GL-m]			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	ROTATING SPEED [rpm]	DRILLING PRESSURE (kpa)	SPT : Standard penetration Test (DISTURB SAMPLE)			
From	To	hh:mm	minute	From	To	Meter drilled										From [GI-m]	To [GI-m]	Total [m]	Numbe time
13:40	13:50	0:10	10	0	0.5	0.5	5	nil	weathered Rock	Brown	19	8	57.89473684	65	1				
14:50	17:00	2:10	130	0.5	1.5	1	0.76923077		weathered Rock	Brown	19	8	57.89473684	65	1				
8:00	10:15	2:15	135	1.5	3	1.5	1.11111111	-	weathered	Brown	20	9	55	150	2				
10:45	13:00	2:15	135	3	4.5	1.5	1.11111111	nil	weathered	Brown	24	11	54.16666667	150	2				
13:40	15:50	2:10	130	4.5	6	1.5	1.15384615	0.27	fractured rock	brown	25	16.8	32.8	65	2				
8:00	12:55	4:55	295	6	7.5	1.5	0.50847458	0.33	fractured rock	brown	10	6	40	65	1				
14:20	17:10	2:50	170	7.5	9	1.5	0.88235294	0.36	fractured rock	brown	18	12.6	30	65	1				
8:00	13:40	5:40	340	9	10.5	1.5	0.44117647	35	fractured rock	black and grey	23	12.8	44.34782609	65					
14:4				10.5	12	1.5	0.78947368	0.2	ractured rock	black and grey	19	19.2	-1.052631579	65					
14:4				12	13.5	1.5	0.83333333	0.76	Fr d	Black and grey	24	16.2	32.5	65					
8:0				13.5	15	1.5	0.53571429	0.8		y	25	16	36	65					
13:40	16:00	2:20	140	15	16.5	1.5	1.07142857	0.4		y	25	16	36	65					
8:00	12:00	4:00	240	16.5	18	1.5	0.625	0.29		y	19	8.6	54.73684211	65	1				
14:00	17:00	3:00	180	18	19.5	1.5	0.83333333	0.33	Fractured	Black and grey	24	16.9	29.58333333	65	1				
8:00	10:10	2:10	130	19.5	21	1.5	1.15384615	0.3	Fractured	Black and grey	20	11.3	43.5	65	1				
10:10	13:15	3:05	185	21	22.5	1.5	0.81081081	0.2	Fractured	Black and grey	23	19.3	16.08695652	65	1				
14:15	15:40	1:25	85	22.5	24	1.5	1.76470588	0.29	Fractured	Black and grey	21	12.6	40	65	1				
15:40	16:40	1:00	60	24	25.5	1.5	2.5	0.3	Fractured	Black and grey	20	18	10	65	1				
16:40	17:40	1:00	60	25.5	27	1.5	2.5	0.23	Fractured	Black and grey	22	12.9	41.36363636	65	1				
8:00	10:00	2:00	120	27	28.5	1.5	1.25	nil	Fractured	Black and grey	20	16.5	17.5	65	1		28.6	0.1	
14:00	17:05	3:05	185	28.5	30	1.5	0.81081081	nil	Fractured	Black and grey	19.9	13.7	31.15577889	65		30	30.05	0.05	
8:00	9:00	1:00	60	30	31.5	1.5	2.5	nil	Fractured	Black and grey	20	12	40	65	1	31.5	31.6	0.1	
10:00	11:05	1:05	65	31.5	33	1.5	2.30769231	0.24	Fractured	Black and grey	23	16.1	30	65	1				
12:10	13:10	1:00	60	33	34.5	1.5	2.5	nil	Fractured	Black and grey	21	17.3	17.61904762	65	1	34.5	34.55	0.05	
13:40	16:40	3:00	180	34.5	36	1.5	0.83333333	nil	Fractured	Black and grey	24	11.9	50.41666667	65	1	36	36.05	0.05	
8:00	9:20	1:20	80	36	37.5	1.5	1.875	nil	Fractured	Black and grey	21	12.3	41.42857143	65	1	37.5	37.55	0.05	
9:50	11:30	1:40	100	37.5	39	1.5	1.5	nil	Fractured	Black and grey	20	11.6	42	65	1	39	39.05	0.05	
11:30	12:50	1:20	80	39	40	1	1.25	0.2	Fractured	Black and grey	21	11.7	44.28571429	65	1				
							1.26747354						36.6						

Stable ground water

Hard rock

Strong leakage zone

Low leakage zone

Leakage increasing!

Interpretation and basis for design

Based on GVAC shown in Appendix-4, groundwater levels showed stable depths after the drillng of GL-4.5m or deeper. Leakage ratio intermittently decreaced at the similar depth and deeper. On the other hand, leakage volume is higher than those of otehr boreholes. Abnormally high leakage ratio was observed between GL-16.5m and 18.0m. Thus the depth of GL-18.0m is considered as the bottom of the loozened zone.

At a depth of 19.5m and deper, leakage increased and groundwater levels were lowered. These suggest further loozened zones in deeper portion.

From the shallower depths, little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. Thus GL-1.4m was set as the stable water level of non-pressured water. In this borehole, low permeable sections lie in the shallower depths. In case of abnormal rainfall, such less permeablesections may work as cap rock (=sealing layer for groundwater) and turn non-pressured groundwater in the loosened zones into pressured one, which may trigger a massive deep-seated landslide.

Appendix

ly Drilling Operation Report

ect Name: Landslide Protection Project

hole Number: ..... BV 9

: 16/03/2021

tion: ..... Barbot ,Pkg. IV-C NH717A

TIME h:mm)		Erapsed time		METERAGE m] [GL-			Drilling speed cm/minute	CORE LENGTH [M]	ROCK NAME [Soil/Highly Weathered/Weathered/Bolder/Rock]	COLOR [Sample and/or Water]	DRILLING BIT	INJECTED WATER [L/MIN]	RETURNED WATER [L/MIN]	Leakage ratio [%]	SPT : Standard penetration Test (DISTURBED SAMPLE)			
h	To	hh:mm	minute	From	To	Meter drilled									From [GL-m]	To [GL-m]	Total [m]	Number of times
0	12:30	0:30	30	0	0.5	0.5	1.66666667	nil	DS Taken									
0	14:00	1:30	90	0.5	1.5	1	1.11111111		weathered Rock	Brown	Diamond	21	13.6	35.2381	1.5	1.6	0.1	70
0	17:00	2:00	120	1.5	3	1.5	1.25	0.2	Hard Rock	Brown	Diamond	24	18.9	21.25				
0	12:40	4:40	280	3	4.5	1.5	0.53571429	0.4	Hard Rock	Brown	Diamond	24	19	20.83333				
0	17:00	3:00	180	4.5	6	1.5	0.83333333	[illegible]65	Hard Rock	Brown	Diamond	19	13.8	27.36842				
0	13:00	5:00	300	6	7.5	1.5	0.5	0.3[illegible]	[illegible]rd Rock	Brown	Diamond	23	17.6	23.47826				
0	17:00	3:00	180	7.5	9	1.5	0.83333333	-	we[illegible]red rock	Brown	Diamond	23	16.9	26.52174				
0	9:00	1:00	60	9	10.5	1.5	2.5	0.5	Hard R[illegible]	Brown	Diamond	24	16.3	32.08333				
0	10:00	1:00	60	10.5	12	1.5	2.5	0.3	Hard [illegible]	[illegible]	[illegible]iamond	20	11.9	40.5				
0	11:00	1:00	60	12	13.5	1.5	2.5	0.37	Hard [illegible]	[illegible]	[illegible]iamond	25.8	19.3	25.1938				
0	12:00	1:00	[illegible]0	13.5	15	1.5	2.5	0.26	Hard [illegible]	[illegible]	[illegible]iamond	19.6	16	18.36735				
0	13:00	1:00	60	15	16.5	1.5	2.5	0.35	Hard Rock	Brown	Diamond	20.8	15.3	26.44231				
				16.5	18	1.5	2.5	0.36	Hard Rock	Brown	Diamond	23	11.9	48.26087				
				18	19.5	1.5	2.5	0.3	Hard Rock	Brown	Diamond	24.9	12.6	49.39759				
				19.5	21	1.5	1.25	0.25	Hard Rock	Brown	Diamond	21	11.1	47.14286				
0	9:00	1:00	60	21	22.5	1.5	2.5	0.26	Hard Rock	Br[illegible]	Diamond	24	12.7	47.08333				
0	10:00	1:00	60	22.5	24	1.5	2.5	nil	weathered rock	Brown	Diamond	21	16.9	19.52381	24	24.05	0.05	58
0	11:00	1:00	60	24	25.5	1.5	2.5	nil	weathered rock	Brown	Diamond	23	12.7	44.78261	25.5	25.55	0.05	60
0	12:00	1:00	60	25.5	27	1.5	2.5	nil	weathered rock	Brown	[illegible]nd	25	18.6	25.6	27	27.05	0.05	60
0	13:00	1:00	60	27	28.5	1.5	2.5	nil	weathered rock	Brown	[illegible]	[illegible]	12.3	35.60209	28.5	28.6	0.1	60
0	17:00	2:30	150	28.5	30	1.5	1	nil	weathered rock	Brown	[illegible]	[illegible]	12.1	42.38095	30	30.05	0.05	60
0	9:00	1:00	60	30	31.5	1.5	2.5	nil	weathered rock	Blackish Gr[illegible]	[illegible]	[illegible]	11.9	46.15385	31.5	31.55	0.05	60
0	10:30	1:30	90	31.5	33	1.5	1.66666667	nil	weathered rock	Blackish Grey	Diamond	23.1	16.3	29.43723	33	33.05	0.05	60
0	11:30	1:00	60	33	34.5	1.5	2.5	nil	weathered rock	Blackish Grey	Diamond	21.5	16.1	25.11628	34.5	34.55	0.05	50
0	12:30	1:00	60	34.5	36	1.5	2.5	nil	weathered rock	Blackish Grey	Diamond	19.9	11.3	43.21608	36	36.05	0.05	60
0	14:00	1:30	90	36	37.5	1.5	1.66666667	nil	weathered rock	Blackish Grey	Diamond	22.5	12.6	44	37.5	37.55	0.05	50
0	15:00	0:50	50	37.5	39	1.5	3	nil	weathered rock	Blackish Grey	Diamond	24	16.9	29.58333				
0	16:40	1:40	100	39	40	1.5	1.5	nil	weathered rock	Blackish Grey	Diamond	24	13.6	43.33333	40	40.05	0.05	50
							1.93976757							34.0				

Stable ground water

Hard rock

Low leakage zone

Strong leakage zone

Hard rock

rpretation and basis for design

d on GVAC shown in Appendix-4, groundwater levels showed stable depths after the drilling of GL-9.0m or deeper. Leakage ratio showed lower value from the surface. On the other d, leakage volume is higher than those of otehr boreholes. Abnormally high leakage ratio was observed from a depth of GL-16.5m and deeper until the bottom of borehole. Drilling speed ests a section of hard rock up to at a depth of 9.0m or shallower. On the other hand, core samples up to a depth of 22.5 meters or shallower are considered as a section of continuous rock. Abnormally high leakage ratio was observed between GL-16.5m and 22.5m.

he purpose of design the depth of GL-22.5m is considered as the bottom of the loozened zone preliminarily. Core samples suggests, however, that the loozened zone between GL-16.5m 22.5m may be the upper boundary of a much larger loozened zone and this borehole may not reach the lower boundary.

depth of 19.5m and deper, leakage increased and groundwater levels were lowered. These suggest further loozened zones in deeper portion.

w the depth of GL-22.5m, little differences between the groundwater levels after the work and before the work of the following working day were observed on GVAC. Thus GL-6.0m was s the stable water level of non-pressured groundwater. In this borehole, low permeable sections lie in the shallower depths. In case of abnormal rainfall, such less permeable sections work as cap rock (=sealing layer for groundwater) and turn non-pressured groundwater in the loosened zones into pressured one, which may trigger a massive deep-seated landslide.

Appendix 4

Groundwater Variation Analysis Chart (GVAC)

Appendix

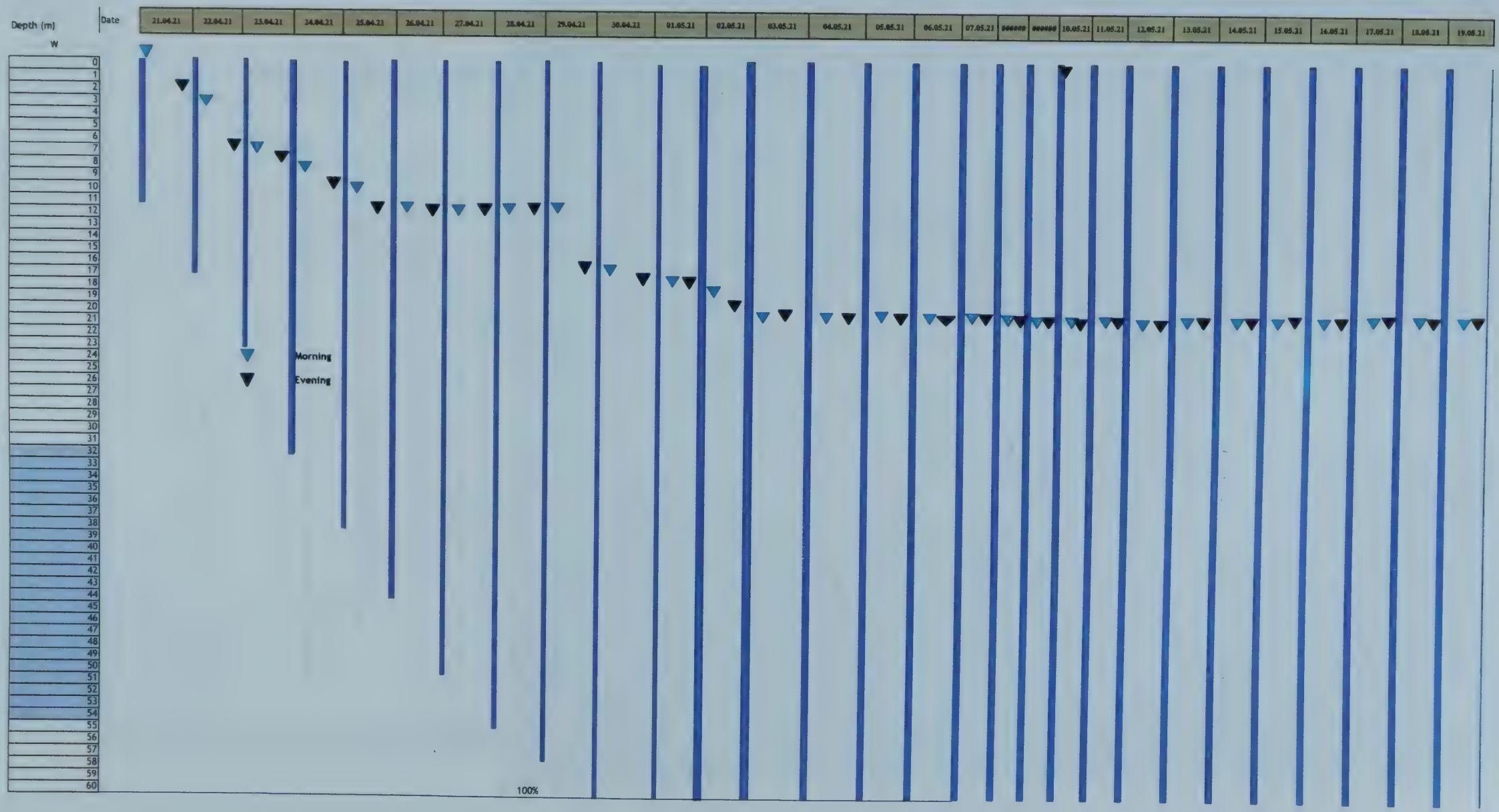
Daily Drilling Operation Report

Project Name : Landslide Protection Project

Borehole Number: ..... BV 1

Location. ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
20.04.21	no groundwater	1.6	10.5
22.04.21	3.9	7.6	16.5
23.04.21	7.8	8.2	22.5
24.04.21	9.1	10.5	31.5
25.04.21	11	12.6	37.5
26.04.21	12.6	12.6	43.5
27.04.21	12.6	12.6	49.5
28.04.21	12.6	12.6	54
29.04.21	12.6	17.5	57
30.04.21	17.7	18.1	60
01.05.21	18.2	18.4	60
02.05.21	19.1	20.3	60
03.05.21	20.3	20.3	60
04.05.21	20.3	20.3	60
05.05.21	20.3	20.3	60
06.05.21	20.3	20.3	60
07.05.21	20.3	20.3	60
08.05.21	20.2	20.3	60
09.05.21	20.3	20.3	60
10.05.21	20.2	20.3	60
11.05.21	20.3	20.3	60
12.05.21	20.3	20.3	60
13.05.21	20.3	20.3	60
14.05.21	20.3	20.3	60
15.05.21	20.3	20.3	60
16.05.21	20.3	20.3	60
17.05.21	20.3	20.3	60
18.05.21	20.3	20.3	60
19.05.21	20.3	20.3	60
20.05.21	20.3	20.3	60
21.05.21	20.3	20.3	60
22.05.21	20.3	20.3	60
23.05.21	20.3	20.3	60
24.05.21	20.1	20.20	60
25.05.21	20.3	20.3	60
26.05.21	20.4	20.3	60
27.05.21	19.7	19.9	60
28.05.21	19.9	20.1	60
29.05.21	20.2	20.2	60
30.05.21	20.2	20.2	60
31.05.21	20.2	20.2	60
01.06.21	20.2	20.3	60
02.06.21	19.8	20.3	60
03.06.21	20.2	20.3	60
04.06.21	20.2	20.3	60
05.06.21	19.8	20.3	60
06.06.21	19.8	19.8	60
07.06.21	19.8	19.8	60
08.06.21	19.8	19.8	60
09.06.21	19.8	19.8	60
10.06.21	19.9	19.8	60
11.06.21	19.9	19.8	60
12.06.21	19.9	19.8	60
13.06.21	19.9	19.8	60
14.06.21	20.3	20.1	60
15.06.21	20.3	20.3	60
16.06.21	20.3	20.3	60
17.06.21	20.3	20.3	60
18.06.21	20.3	20.3	60
19.06.21	19.6	19.6	60
20.06.21	19.6	19.6	60
21.06.21	20.3	20.3	60
22.06.21	20.3	20.3	60
23.06.21	20.3	20.3	60
24.06.21	20.3	20.3	60
25.06.21	20.3	20.3	60
26.06.21	20.3	20.3	60
27.06.21	20.3	20.3	60
28.06.21	20.3	20.3	60
29.06.21	20.3	20.3	60



A04-33

Appendix

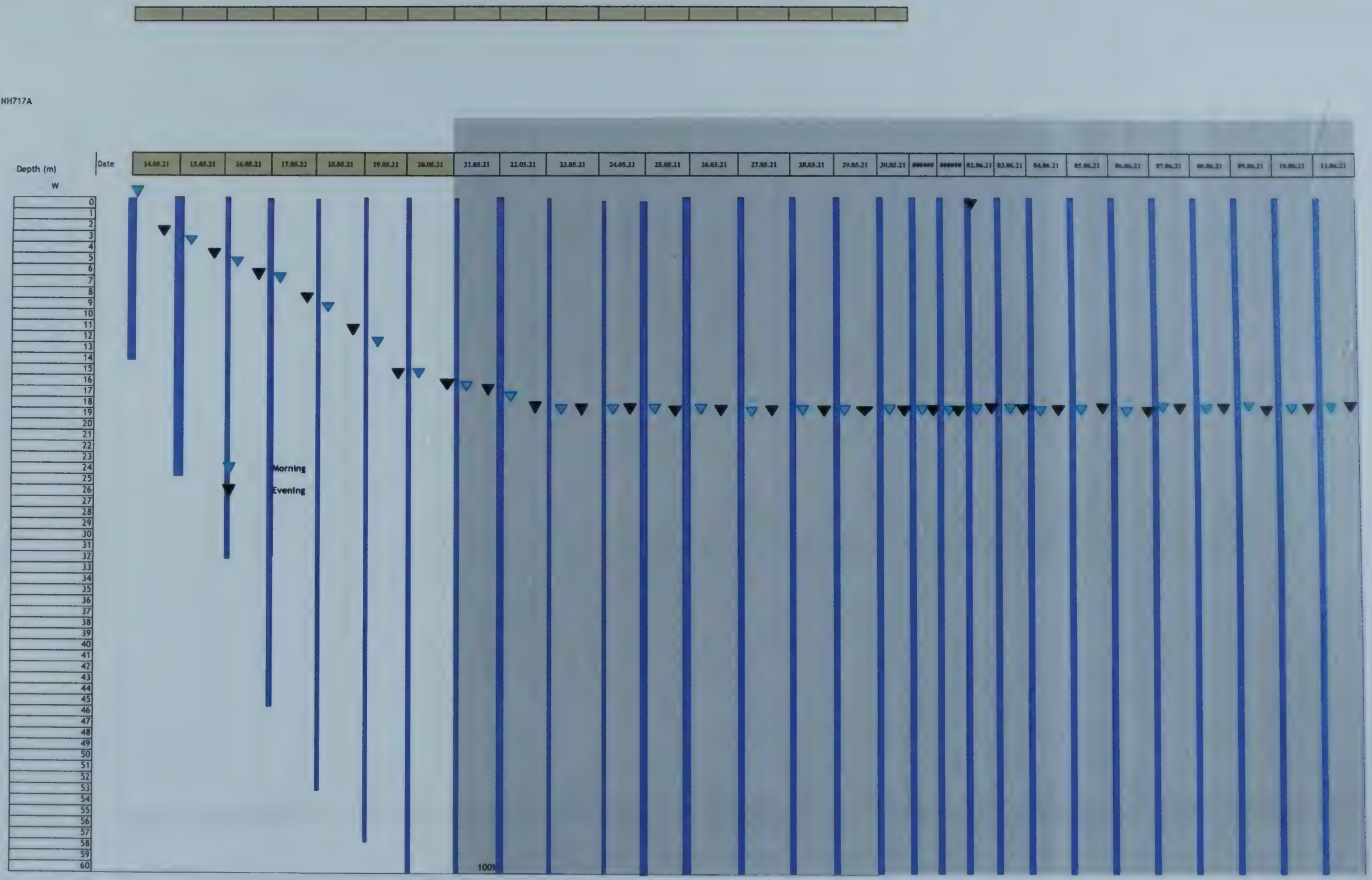
Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 2

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
14.05.21	no groundwater	3.2	13.5
[illegible]	4.3	5.6	24
[illegible]	6.1	7.3	31.5
[illegible]	7.9	9.2	45
[illegible]	10.2	17.1	52.5
[illegible]	13.5	16.1	57
[illegible]	16.2	17.3	60
[illegible]	17.6	17.9	60
[illegible]	18.5	18.5	60
[illegible]	18.5	18.5	60
[illegible]	18.5	18.5	60
25.05.21	18.5	18.5	60
26.05.21	18.5	18.5	60
27.05.21	18.5	18.5	60
[illegible]	18.5	18.5	60
29.05.21	18.5	18.5	60
30.05.21	18.5	18.5	60
31.05.21	18.5	18.5	60
01.06.21	18.5	18.5	60
02.06.21	18.5	18.5	60
03.06.21	18.2	18.3	60
04.06.21	18.3	18.3	60
05.06.21	18.2	18.2	60
06.06.21	18.3	18.3	60
07.06.21	18.5	18.5	60
08.06.21	18.5	18.5	60
09.06.21	18.5	18.5	60
10.06.21	18.5	18.5	60
11.06.21	18.4	18.4	60
12.06.21	18.4	18.4	60
13.06.21	18.4	18.4	60
14.06.21	18.4	18.4	60
15.06.21	18.4	18.4	60
16.06.21	18.4	18.4	60
17.06.21	18.4	18.4	60
18.06.21	18.4	18.4	60
19.06.21	18.4	18.4	60
20.06.21	18.4	18.4	60
21.06.21	18.4	18.4	60
22.06.21	18.4	18.4	60
23.06.21	18.2	18.2	60
24.06.21	18.2	18.2	60
25.06.21	18.2	18.2	60
26.06.21	18.2	18.2	60
27.06.21	18.2	18.2	60
28.06.21	18.2	18.2	60
29.06.21	18.2	18.2	60
30.06.21	18.2	18.2	60
01.07.21	18.2	18.2	60
02.07.21	18.2	18.2	60
03.07.21	18.2	18.2	60
04.07.21	18.2	18.2	60
05.07.21	18.2	18.2	60



A04-34

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 3

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
26.05.21	no groundwater	1.6	9
27.05.21	2.7	4.1	15
28.05.21	5.2	9.7	24
29.05.21	9.9	12.6	28.5
30.05.21	13.2	17.9	34.5
31.05.21	18	21.5	37.5
01.06.21	22.5	22.6	40
02.06.21	22.6	22.6	40
03.06.21	22.6	22.6	40
04.06.21	22.6	22.6	40
05.06.21	22.6	22.6	40
06.06.21	22.6	22.6	40
07.06.21	22.6	22.6	40
08.06.21	22.6	22.6	40
09.06.21	22.6	22.6	40
10.06.21	22.6	22.6	40
11.06.21	22.4	22.4	40
12.06.21	22.4	22.4	40
13.06.21	22.4	22.4	40
14.06.21	22.4	22.4	40
15.06.21	22.4	22.4	40
16.06.21	22.4	22.4	40
17.06.21	22.4	22.4	40
18.06.21	22.4	22.4	40
19.06.21	22.4	22.4	40
20.06.21	22.4	22.4	40
21.06.21	22.4	22.4	40
22.06.21	22.4	22.4	40
23.06.21	22.4	22.4	40
24.06.21	22.4	22.4	40
25.06.21	22.4	22.4	40
26.06.21	22.4	22.4	40
27.06.21	22.4	22.4	40
28.06.21	22.4	22.4	40
29.06.21	22.4	22.4	40
30.06.21	22.4	22.4	40
01.07.21	22.4	22.4	40
02.07.21	22.4	22.4	40
03.07.21	22.4	22.4	40
04.07.21	22.4	22.4	40
05.07.21	22.4	22.4	40

Depth (m) W
0
1
2
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4
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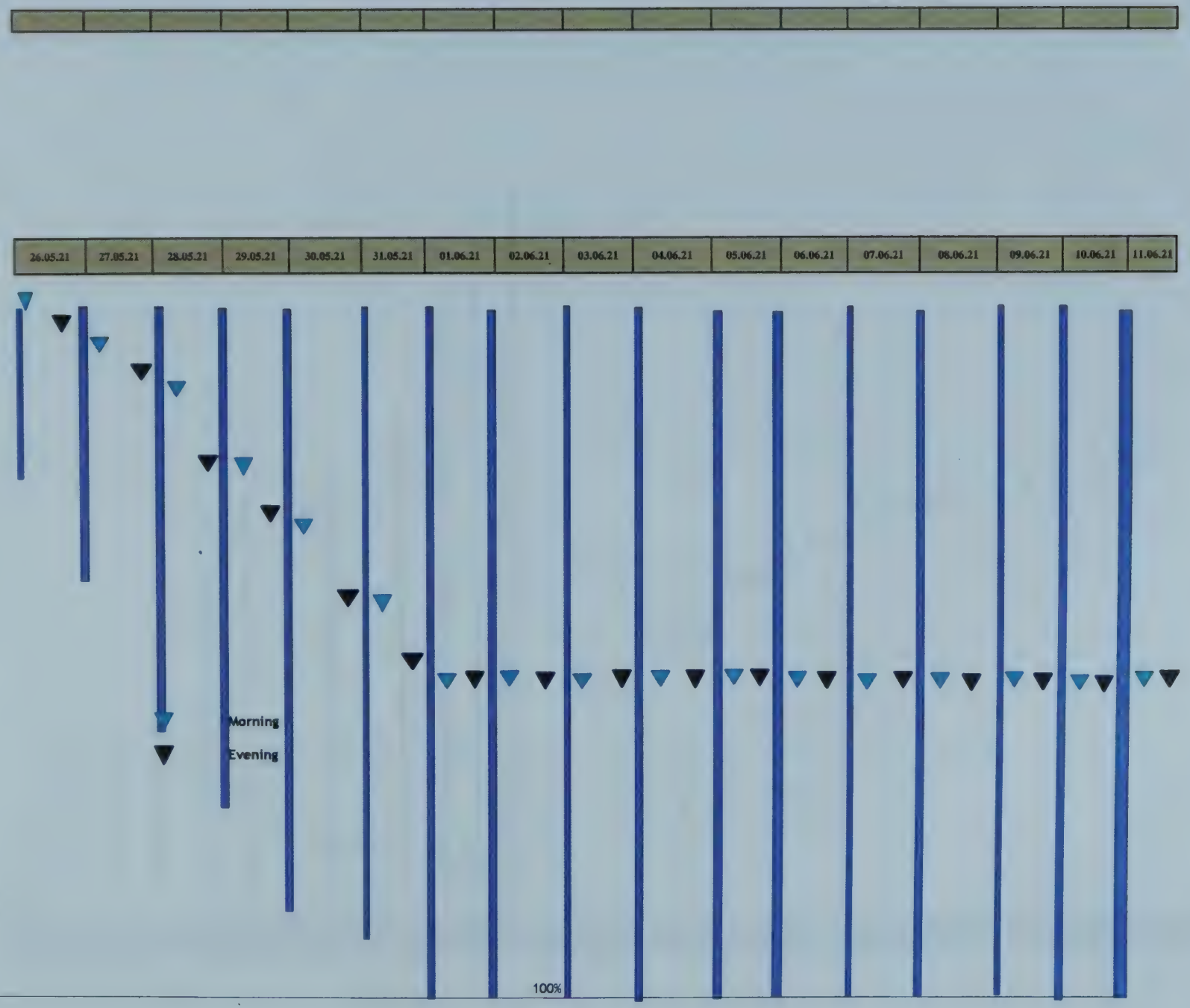

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 4

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
26.1.21	No Groundwater	No Groundwater	1.5
27.1.21	No Groundwater	No Groundwater	7.5
28.1.21	No Groundwater	No Groundwater	9
29.1.21	No Groundwater	7.10	13.5
30.1.21	8.3	6.20	17
31.1.21	8.8	4.30	22
1.2.21	9	3.10	24
2.2.21	9.15	3.05	24
3.2.21	9.3	2.90	24
4.2.21	9.4	2.80	28.5
5.2.21	9.2	3.00	34.5
6.2.21	9.9	3.60	37.5
7.2.21	12.5	4.10	40
8.2.21	12.5	12.50	40
9.2.21	12.5	12.50	40
10.2.21	14.9	14.90	40
11.2.21	15.3	15.60	40
12.2.21	18.8	18.90	40
13.2.21	19.5	20.00	40
14.2.21	20.5	20.90	40
15.2.21	22.7	22.70	40
16.2.21	23.1	23.10	40
17.2.21	23.8	23.85	40
18.2.21	24.3	24.35	40
19.2.21	24.45	24.45	40
20.2.21	24.6	24.65	40
21.2.21	25.8	25.85	40
22.2.21	25.85	25.88	40
23.2.21	26.9	26.90	40
24.2.21	26.9	26.90	40
25.2.21	26.9	26.90	40
26.2.21	26.9	26.90	40
27.2.21	26.9	26.90	40
28.2.21	27	27.00	40
1.3.21	27	27.00	40
2.3.21			
3.3.21			
4.3.21			
5.3.21			
6.3.21	The pipe damaged by local in the night and hole got filled with mud.		
7.3.21			
8.3.21			

Depth (m)	W
0	
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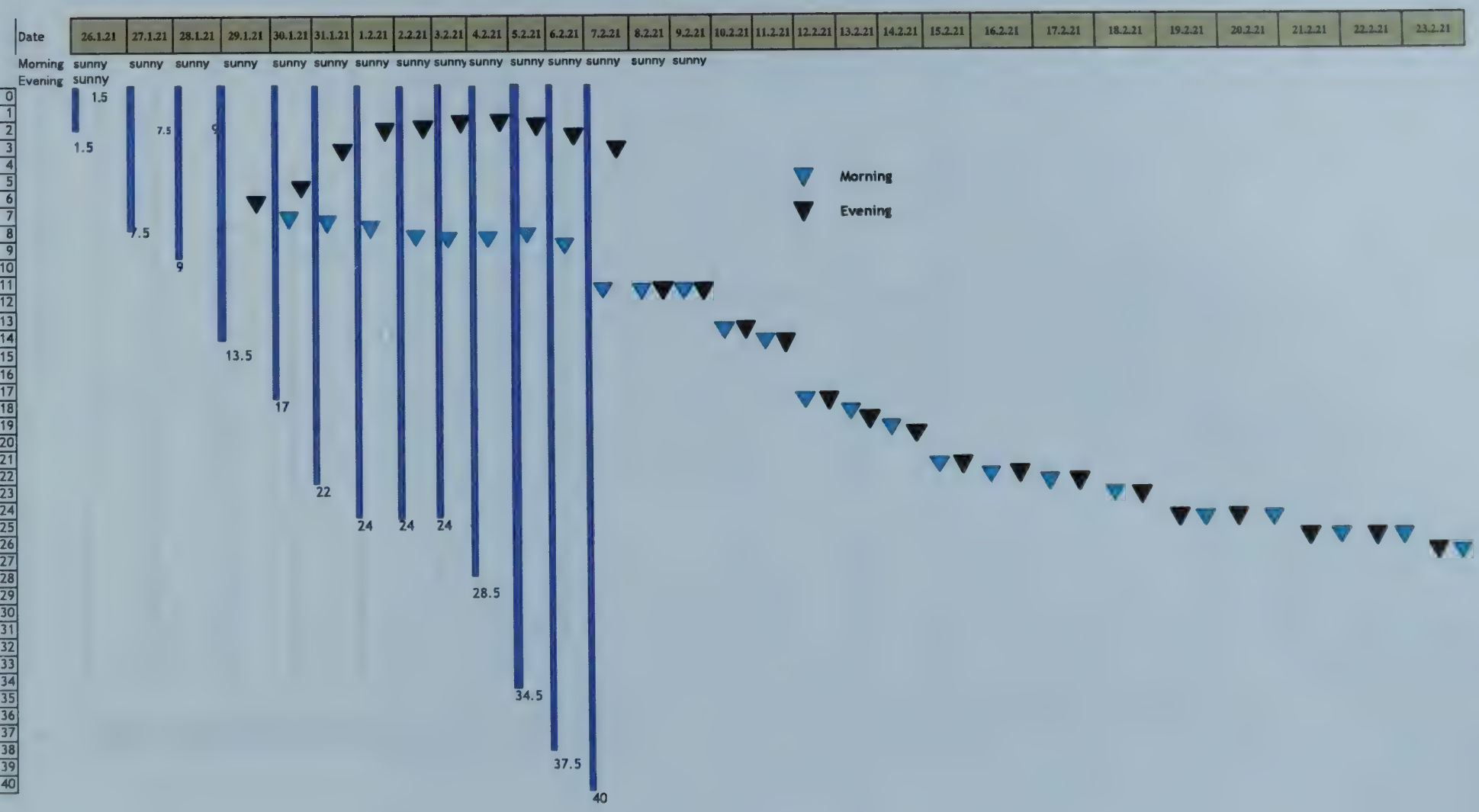


A04-36

Appendix

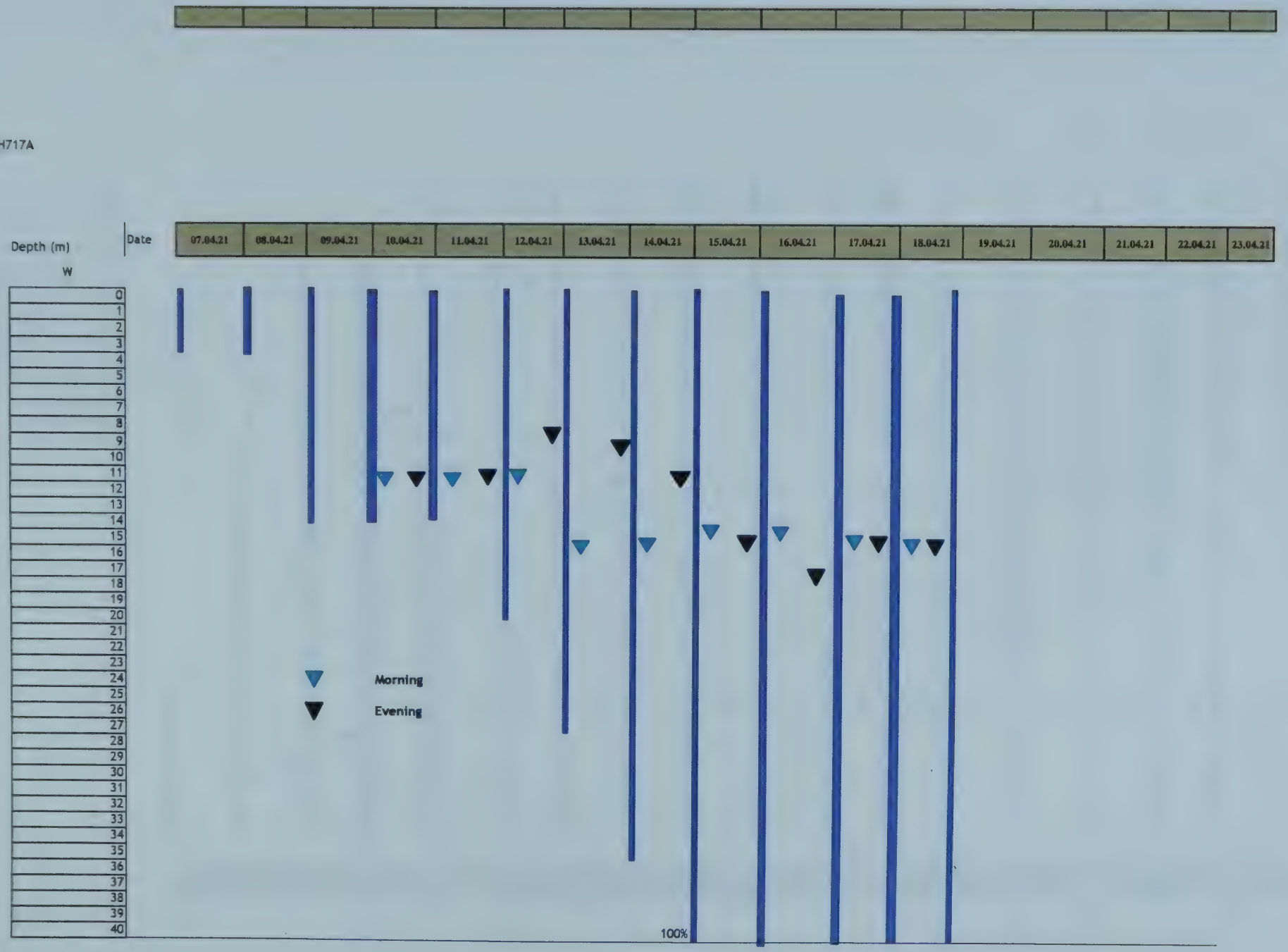
Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 5

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
07.04.21	no groundwater	no groundwater	3
08.04.21	no groundwater	no groundwater	3
09.04.21	no groundwater	no groundwater	13.5
10.04.21	12.1	12.1	13.5
11.04.21	12.1	12.1	13.5
12.04.21	12.1	9.6	19.5
13.04.21	16.5	10.3	25.5
14.04.21	16.3	12.1	34.5
15.04.21	15.4	16.3	40
16.04.21	15.6	18.5	40
17.04.21	16.1	16.3	40
18.04.21	16.5	16.5	40
19.04.21	16.5	16.5	40
20.04.21	16.5	16.5	40
21.04.21	16.5	16.5	40
22.04.21	16.5	16.5	40
23.04.21	16.5	16.5	40
24.04.21	16.5	16.5	40
25.04.21	16.5	16.5	40
26.04.21	16.5	16.5	40
27.04.21	16.5	16.5	40
28.04.21	16.5	16.5	40
29.04.21	16.5	16.5	40
30.04.21	16.5	16.5	40
01.05.21	16.5	16.5	40
02.05.21	16.5	16.5	40
03.05.21	16.5	16.5	40
04.05.21	16.5	16.5	40
05.05.21	16.5	16.5	40
06.05.21	16.5	16.5	40
07.05.21	16.5	16.5	40
08.05.21	16.5	16.5	40
09.05.21	16.5	16.5	40
10.05.21	16.5	16.5	40
11.05.21	16.5	16.5	40
12.05.21	16.5	16.5	40
13.05.21	16.5	16.5	40
14.05.21	16.5	16.5	40
15.05.21	16.5	16.5	40
16.05.21	16.5	16.5	40
17.05.21	16.5	16.5	40
18.05.21	16.5	16.5	40
19.05.21	16.5	16.5	40
20.05.21	16.5	16.5	40
21.05.21	16.5	16.5	40
22.05.21	16.5	16.5	40
23.05.21	16.5	16.5	40
24.05.21	16.3	16.3	40
25.05.21	16.3	16.3	40
26.05.21	16.3	16.3	40
27.05.21	16.3	16.3	40
28.05.21	16.3	16.3	40



A04-37

Appendix

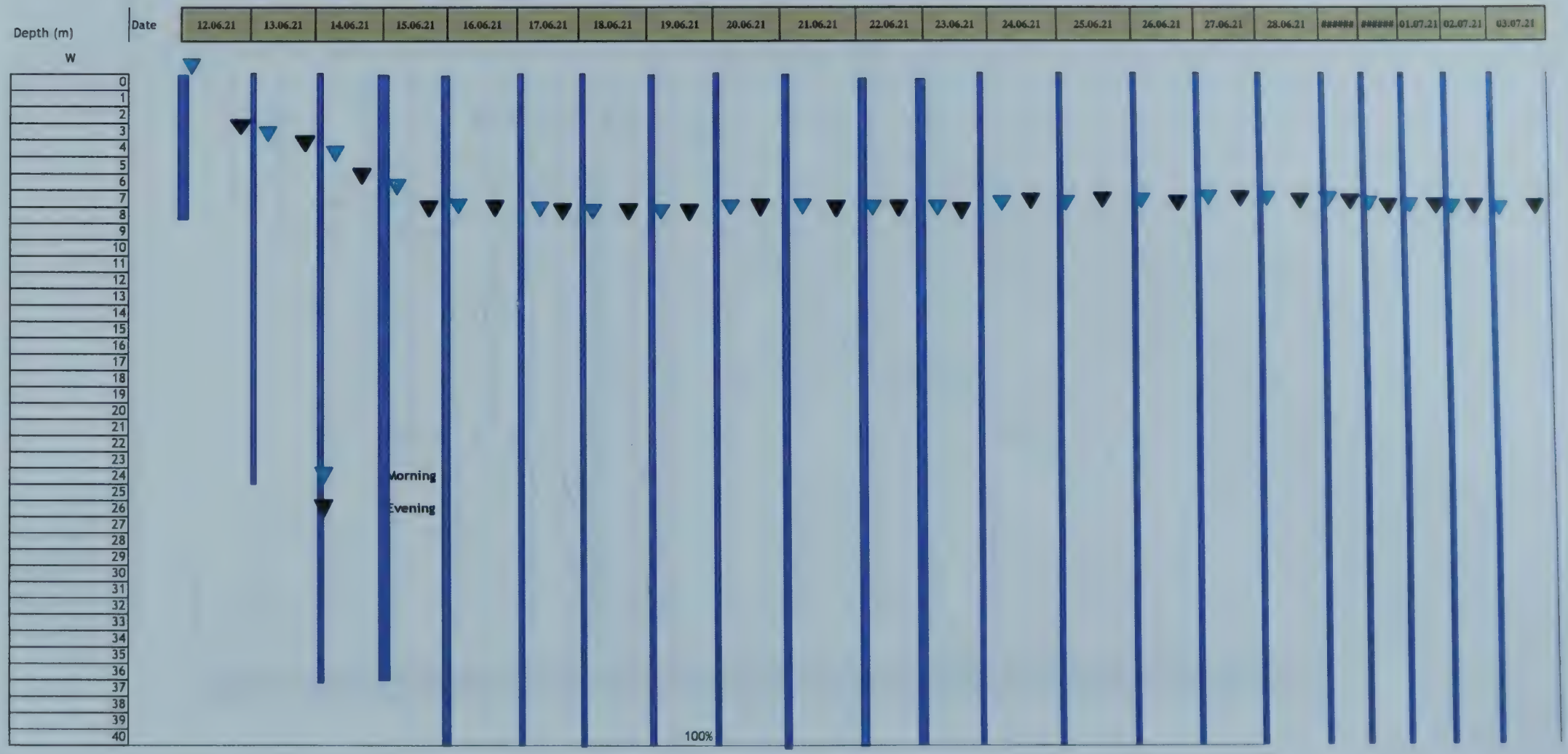
Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 6

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
12.06.21	no groundwater	3.9	7.5
13.06.21	4.1	4.9	24
14.06.21	5.2	6.9	36
15.06.21	7.1	8.5	36
16.06.21	8.5	8.5	40
17.06.21	8.5	8.5	40
18.06.21	8.5	8.5	40
19.06.21	8.5	8.5	40
20.06.21	8.5	8.5	40
21.06.21	8.5	8.5	40
22.06.21	8.5	8.5	40
23.06.21	8.5	8.5	40
24.06.21	8.2	8.2	40
25.06.21	8.2	8.2	40
26.06.21	8.2	8.2	40
27.06.21	8.2	8.2	40
28.06.21	8.2	8.2	40
29.06.21	8.2	8.5	40
30.06.21	8.5	16.5	40
01.07.21	8.5	16.5	40
02.07.21	8.5	16.5	40
03.07.21	8.5	16.5	40
04.07.21	8.5	16.5	40
05.07.21	8.5	16.5	40



A04-38

Appendix

Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 7

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
06.06.21	no groundwater	5.3	9
07.06.21	6.8	7.1	19.5
08.06.21	7.8	8.4	33
09.06.21	8.5	9.4	40
10.06.21	9.9	11.6	40
11.06.21	12.5	12.5	40
12.06.21	12.5	12.5	40
13.06.21	12.5	12.5	40
14.06.21	12.5	12.5	40
15.06.21	12.5	12.5	40
16.06.21	12.5	12.5	40
17.06.21	12.5	12.5	40
18.06.21	12.5	12.5	40
19.06.21	12.5	12.5	40
20.06.21	12.5	12.5	40
21.06.21	12.5	12.5	40
22.06.21	12.5	12.5	40
23.06.21	12.5	12.5	40
24.06.21	12.1	12.1	40
25.06.21	12.1	12.1	40
26.06.21	12.1	12.1	40
27.06.21	12.1	12.1	40
28.06.21	12.1	12.1	40
29.06.21	12.1	12.1	40
30.06.21	12.1	12.1	40
01.07.21	12.5	12.5	40
02.07.21	12.5	12.5	40
03.07.21	12.5	12.5	40
04.07.21	12.5	12.5	40
05.07.21	12.5	12.5	40

Depth (m) w
0
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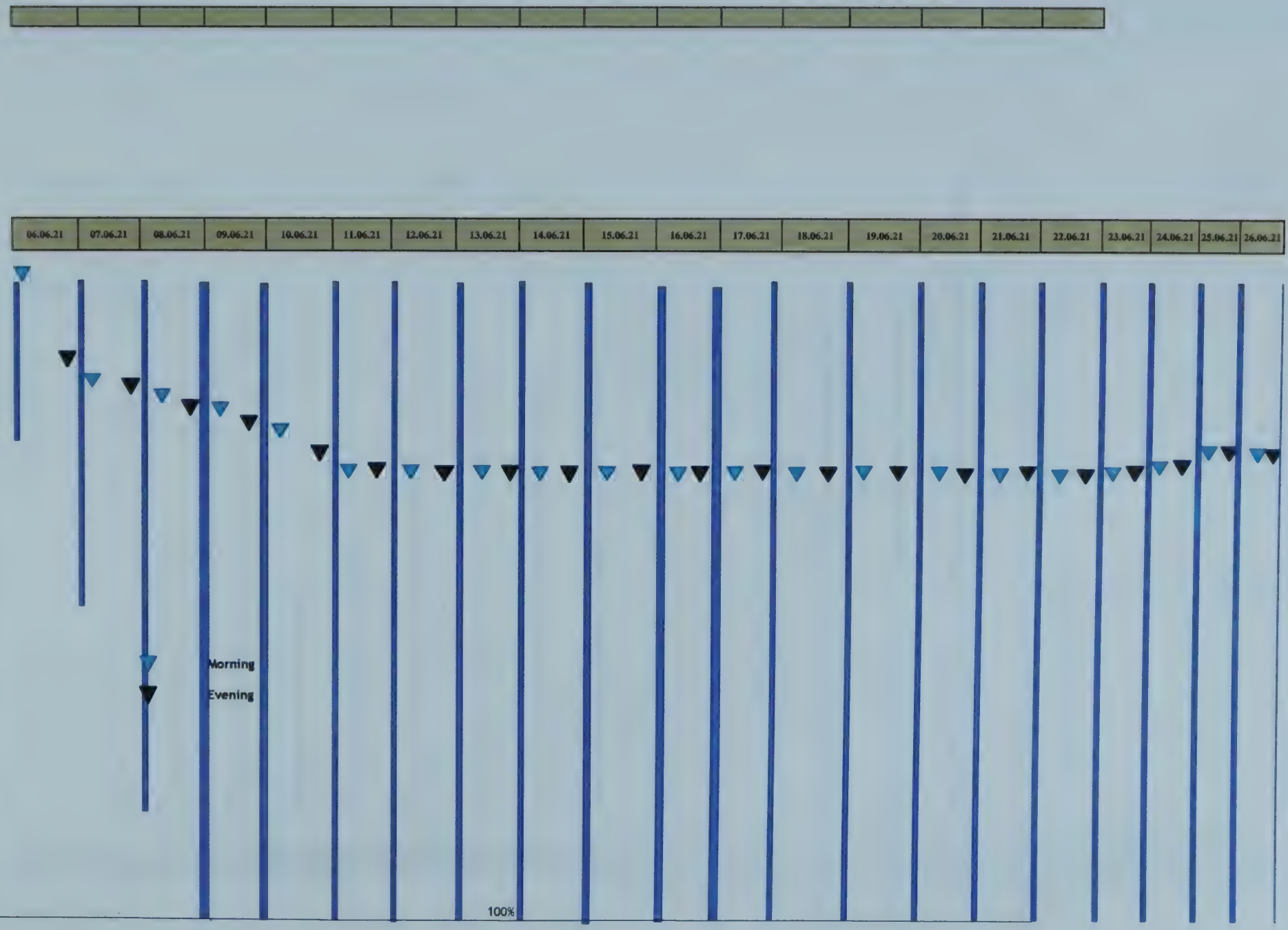


A04-39

Appendix

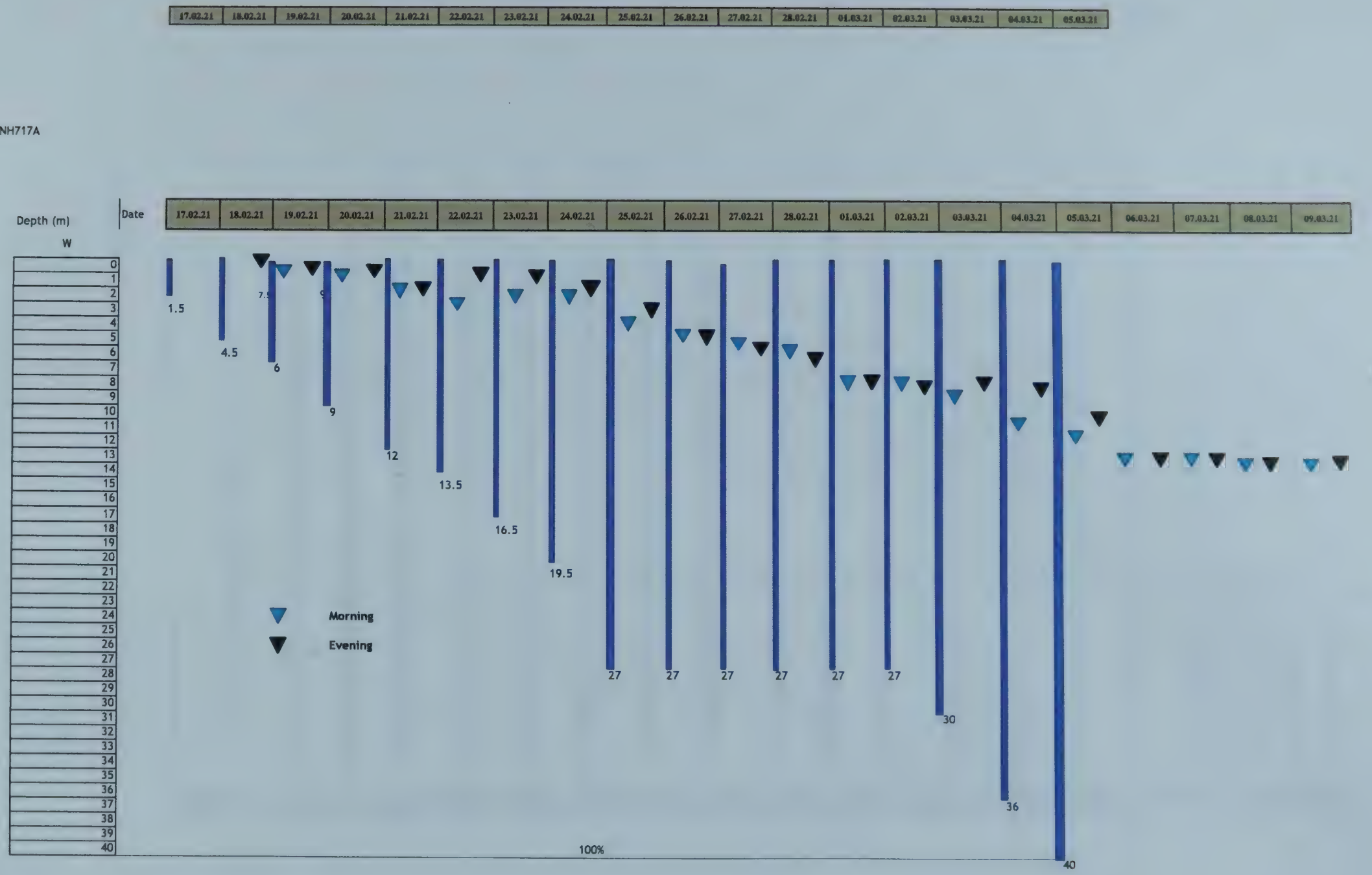
Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 8

Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
17.02.21	No Groundwater	No Groundwater	1.5
18.02.21	No Groundwater	0.8	4.5
19.02.21	1.4	1.1	6
20.02.21	1.6	1.2	9
21.02.21	2.9	2.5	12
22.02.21	3.5	1.2	13.5
23.02.21	3	1.8	16.5
24.02.21	3	2.5	19.5
25.02.21	5	4	27
26.02.21	5.7	5.9	27
27.02.21	6.1	6.5	27
28.02.21	6.9	7.2	27
01.03.21	9	9	27
02.03.21	9	9.1	27
03.03.21	9.9	9	30
04.03.21	11.7	9.2	36
05.03.21	12.6	11.3	40
06.03.21	14.1	14.10	40
07.03.21	14.1	14.10	40
08.03.21	14.2	14.20	40
09.03.21	14.3	14.30	40
10.03.21	14.5	14.50	40
11.03.21	14.5	14.50	40
12.03.21	14.5	14.50	40
13.03.21	14.5	14.50	40
14.03.21	14.5	14.50	40
15.03.21	14.5	14.50	40
16.03.21	14.5	14.50	40
17.03.21	14.5	14.50	40
18.03.21	14.5	14.50	40
19.03.21	14.5	14.50	40
20.03.21	14.5	14.50	40
21.03.21	14.5	14.50	40
22.03.21	14.5	14.50	40
23.03.21	14.5	14.50	40
24.03.21	14.5	14.50	40
25.03.21	14.5	14.50	40
26.03.21	14.5	14.50	40
27.03.21	14.5	14.50	40
28.03.21	14.5	14.50	40
29.03.21	14.5	14.50	40
30.03.21	14.5	14.50	40
31.03.21	14.5	14.50	40
01.04.21	14.5	14.50	40
02.04.21	14.5	14.50	40
03.04.21	14.5	14.50	40
04.04.21	14.5	14.50	40
05.04.21	14.5	14.50	40
06.04.21	14.5	14.50	40
07.04.21	14.5	14.50	40
08.04.21	14.5	14.50	40
09.04.21	14.5	14.50	40



A04-40

Appendix

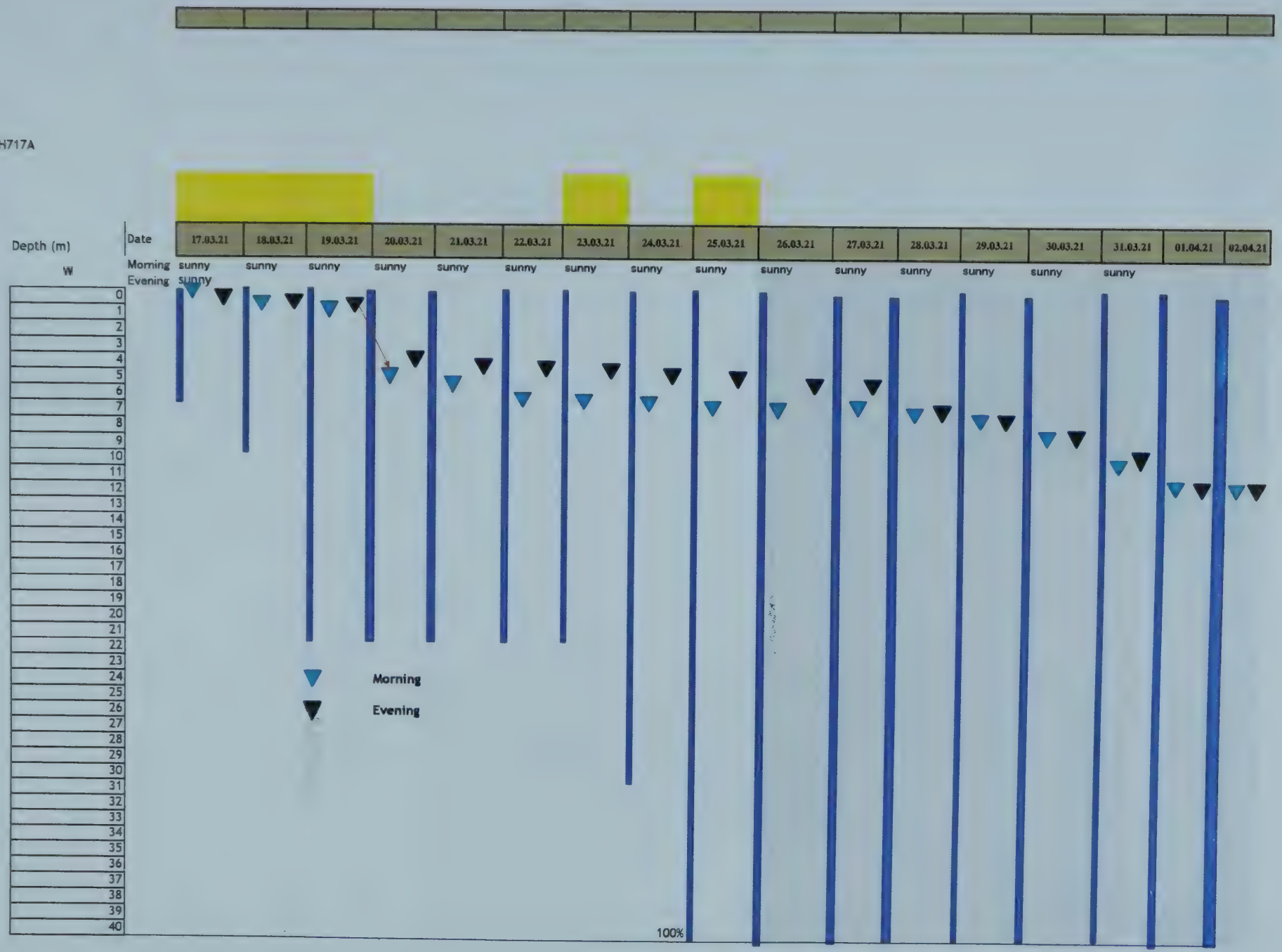
Daily Drilling Operation Report

Project Name: Landslide Protection Project

Borehole Number: ..... BV 9

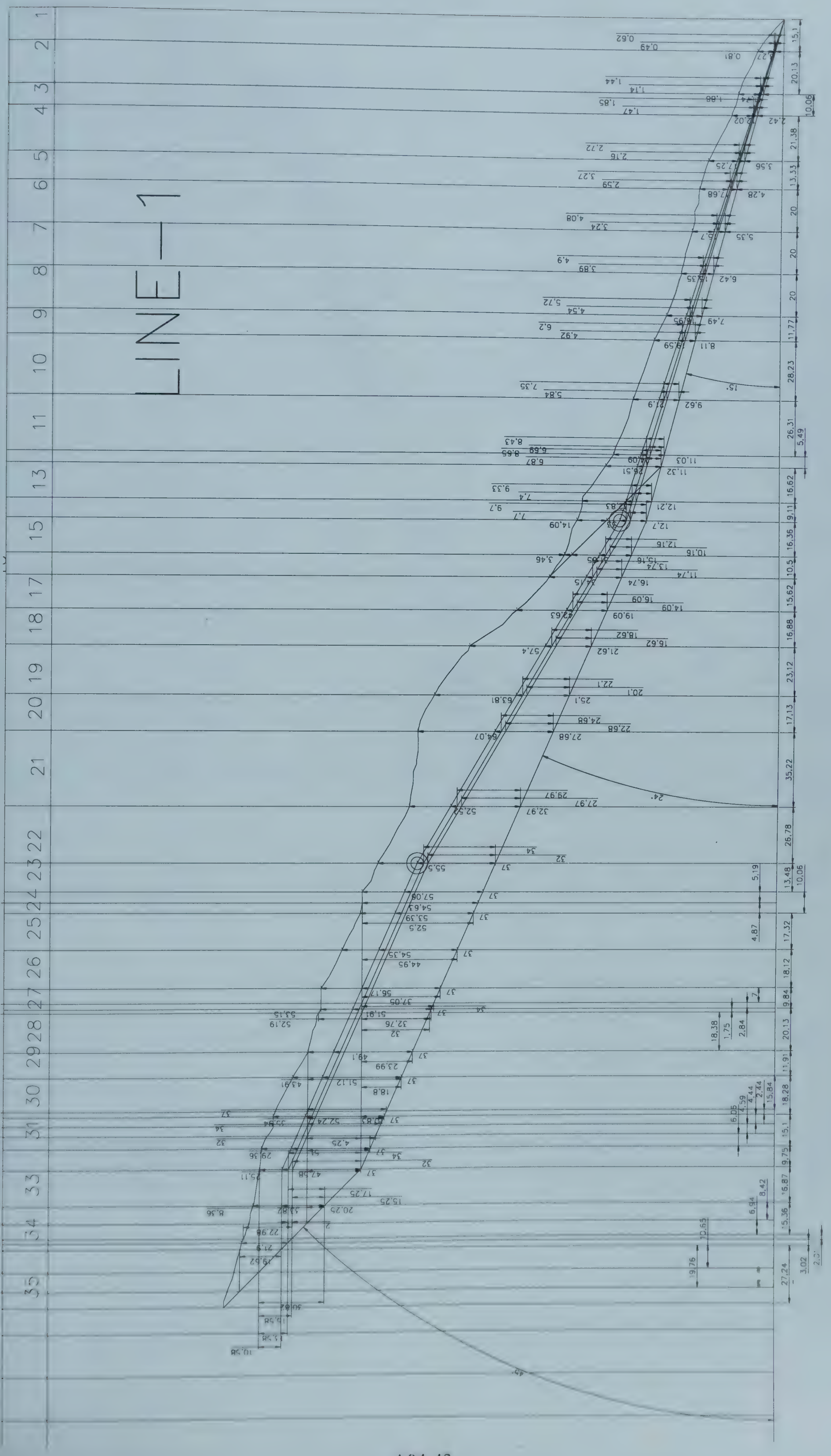
Location: ..... Barbot ,Pkg. IV-C NH717A

Date	GW level from Top (m) Morning	GW level from Top (m) Evening	Drilling Depth
17.03.21	0.7	1	6
18.03.21	1.4	1.3	9
19.03.21	2	1.5	21
20.03.21	6	4.9	21
21.03.21	6.5	5.1	21
22.03.21	7.1	5.3	21
23.03.21	7.5	5.6	21
24.03.21	7.7	5.9	30
25.03.21	7.8	6	40
26.03.21	7.9	6.3	40
27.03.21	7.9	6.4	40
28.03.21	8.2	8	40
29.03.21	8.5	8.5	40
30.03.21	9.7	9.7	40
31.03.21	11.2	10.9	40
01.04.21	12.7	12.7	40
02.04.21	12.7	12.7	40
03.04.21	12.7	12.7	40
04.04.21	12.7	12.7	40
05.04.21	12.7	12.7	40
06.04.21	12.7	12.7	40
07.04.21	12.7	12.7	40
08.04.21	12.7	12.7	40
09.04.21	12.7	12.7	40
10.04.21	12.7	12.7	40
11.04.21	12.7	12.7	40
12.04.21	12.7	12.7	40
13.04.21	12.7	12.7	40
14.04.21	12.7	12.7	40
15.04.21	12.7	12.7	40
16.04.21	12.7	12.7	40
17.04.21	12.7	12.7	40
18.04.21	12.7	12.7	40
19.04.21	12.7	12.7	40
20.04.21	12.7	12.7	40
21.04.21	12.7	12.7	40
22.04.21	12.7	12.7	40
23.04.21	12.7	12.7	40
24.04.21	12.7	12.7	40
25.04.21	12.7	12.7	40
26.04.21	12.7	12.7	40
27.04.21	12.7	12.7	40
28.04.21	12.7	12.7	40
29.04.21	12.7	12.7	40
30.04.21	12.7	12.7	40
01.05.21	12.7	12.7	40
02.05.21	12.7	12.7	40
03.05.21	12.7	12.7	40
04.05.21	12.7	12.7	40
05.05.21	12.7	12.7	40
06.05.21	12.7	12.7	40
07.05.21	12.7	12.7	40



A04-41

LINE 1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2,5°
15°
24°
45°
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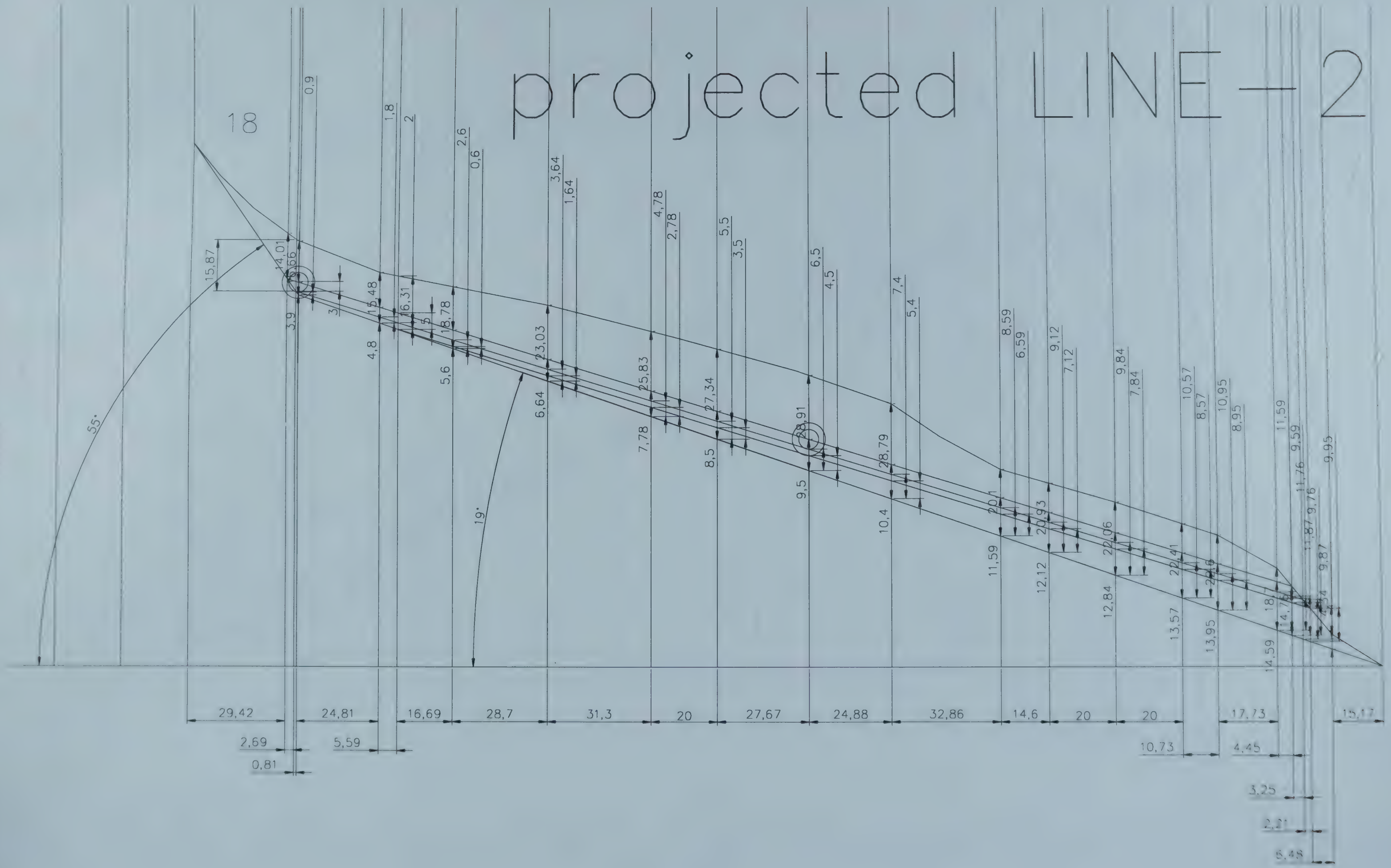
10,58

projected LINE 1-2

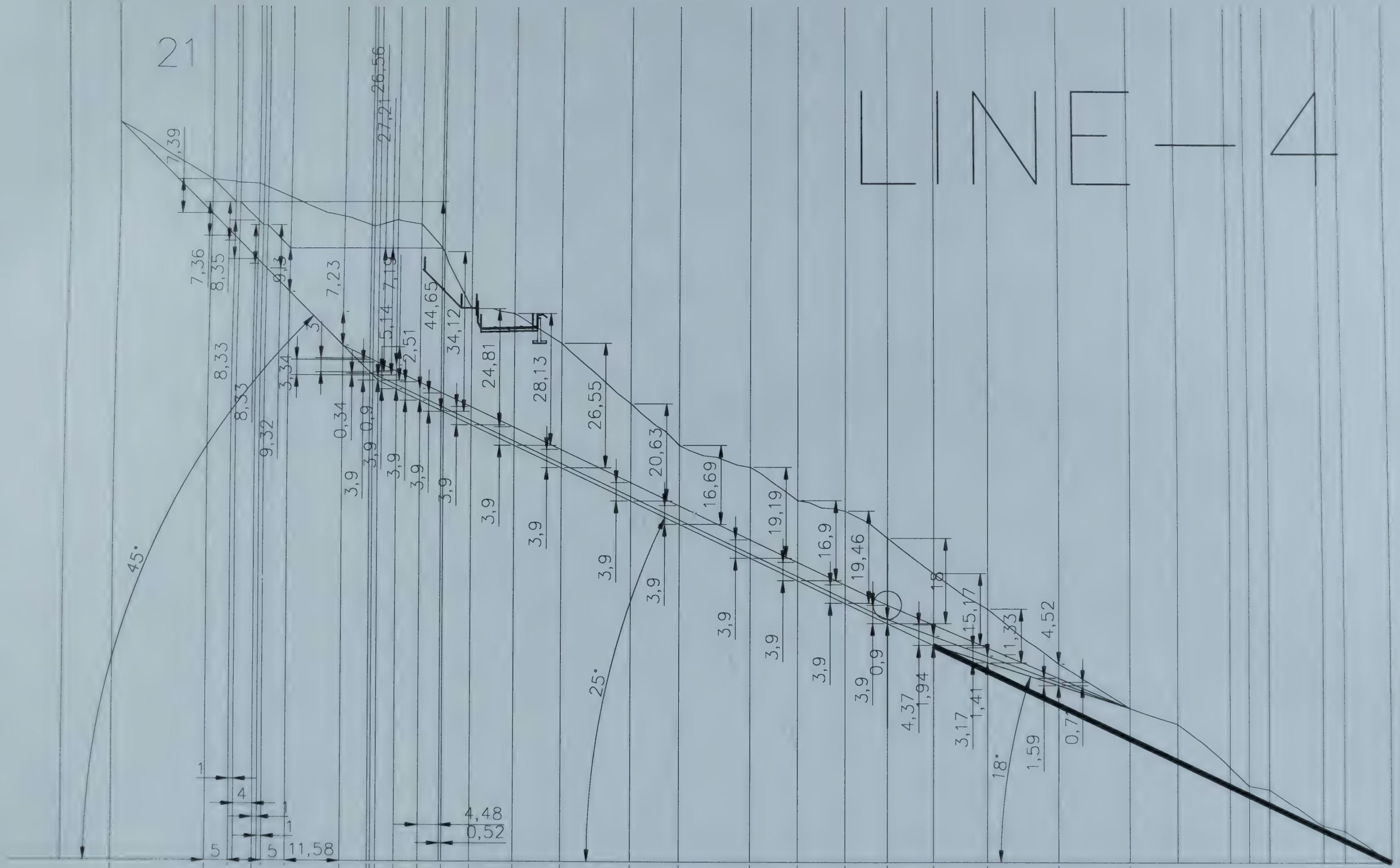


A04-44

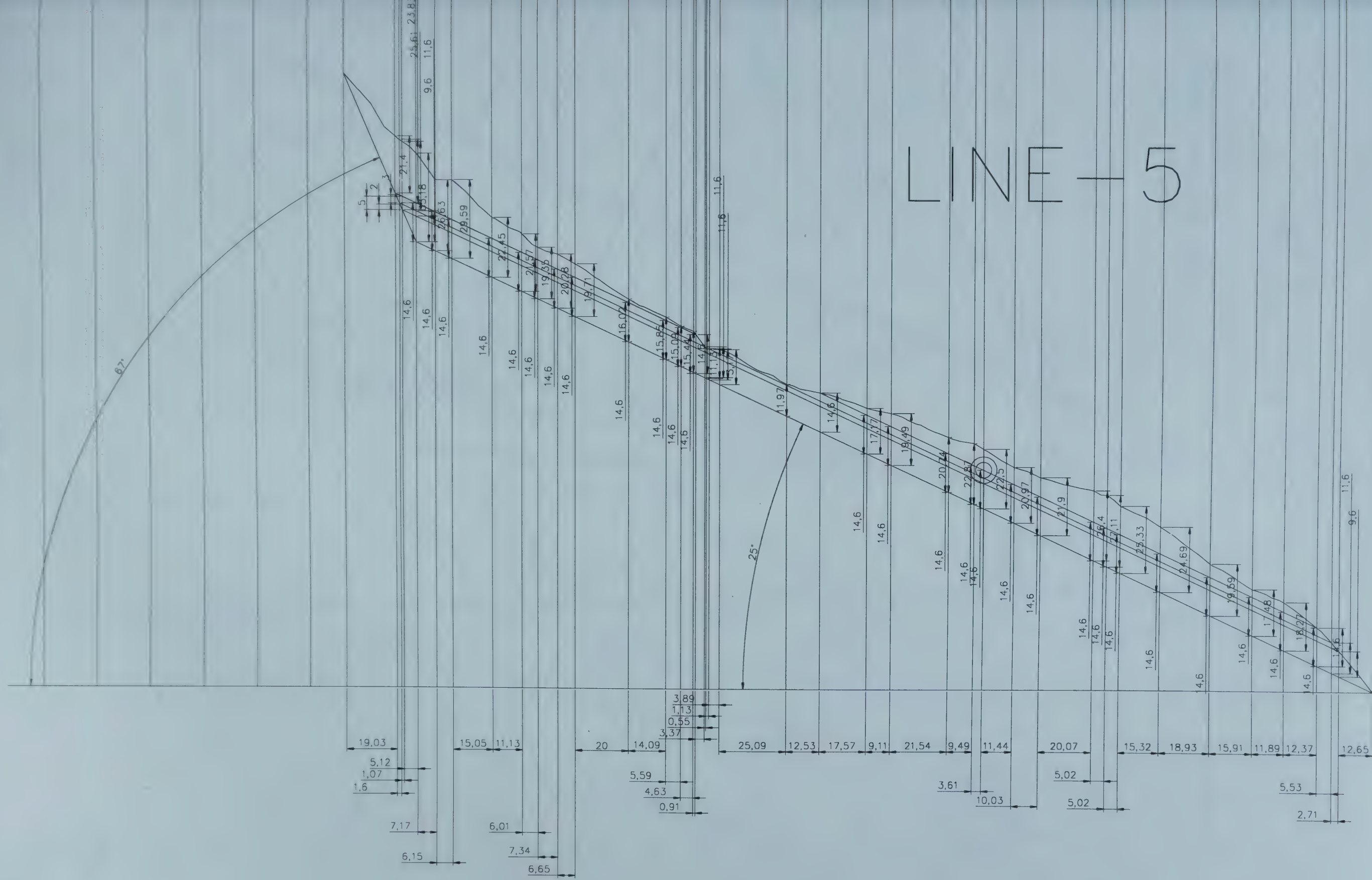
LINE-3

A04-45

LINE 14



A04-46


LINE-5
67°
25°
19,03
5,12
1,07
1,6
7,17
6,15
15,05
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0,91
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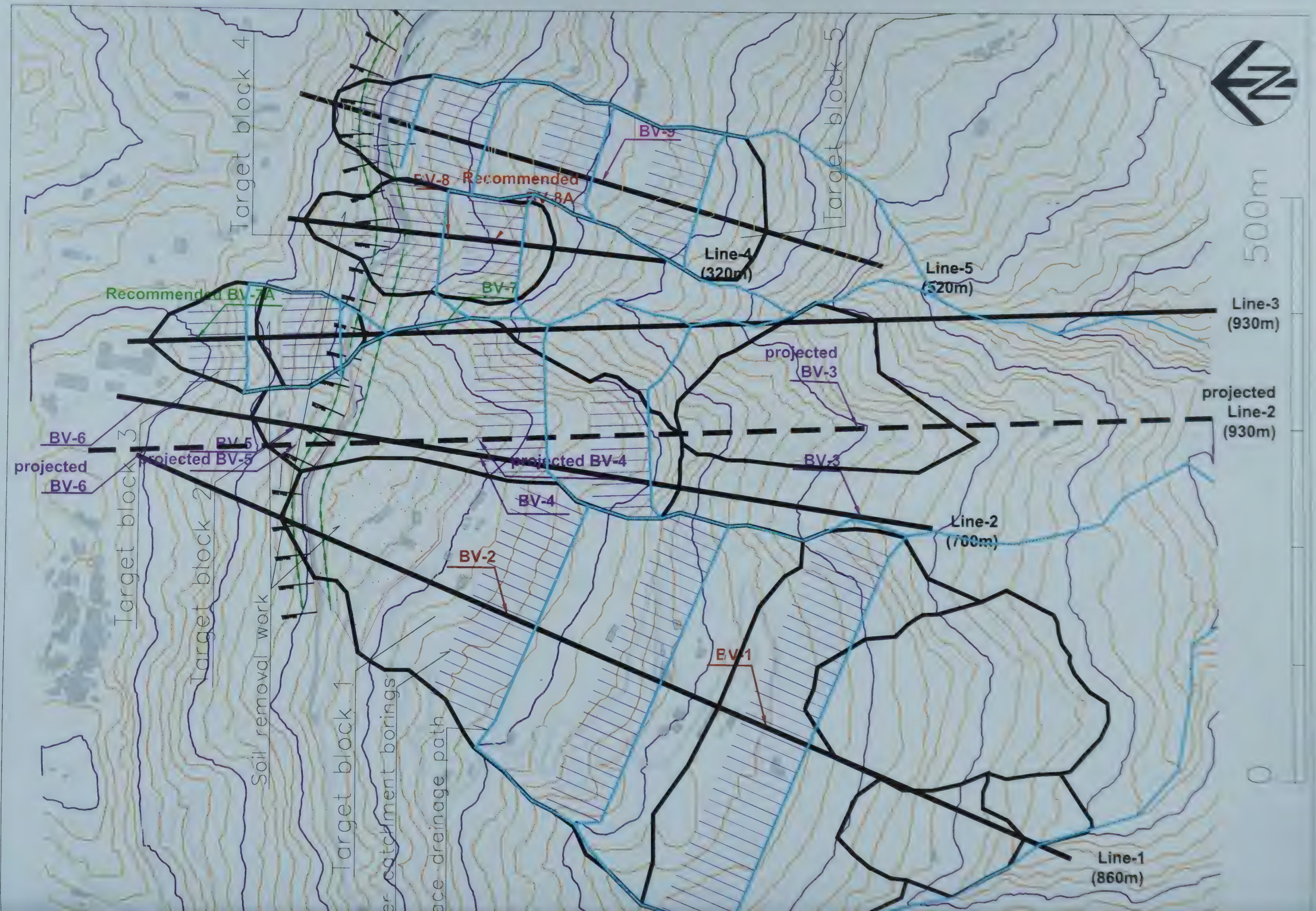
2,71

A04-47

Appendix 6

Image of Control Works



Target block 4
Target block 5
Target block 3
Target block 2
Target block 1
Soil removal work
catchment borings
drainage path
BV-9
BV-8
Recommended BV-8A
BV-7
Recommended BV-7A
Line-4 (320m)
Line-5 (520m)
Line-3 (930m)
projected BV-3
projected Line-2 (930m)
BV-6
projected BV-6
BV-5
projected BV-5
projected BV-4
BV-3
BV-4
Line-2 (700m)
BV-2
BV-1
Line-1 (860m)
500m
0

A04-49

A04-50

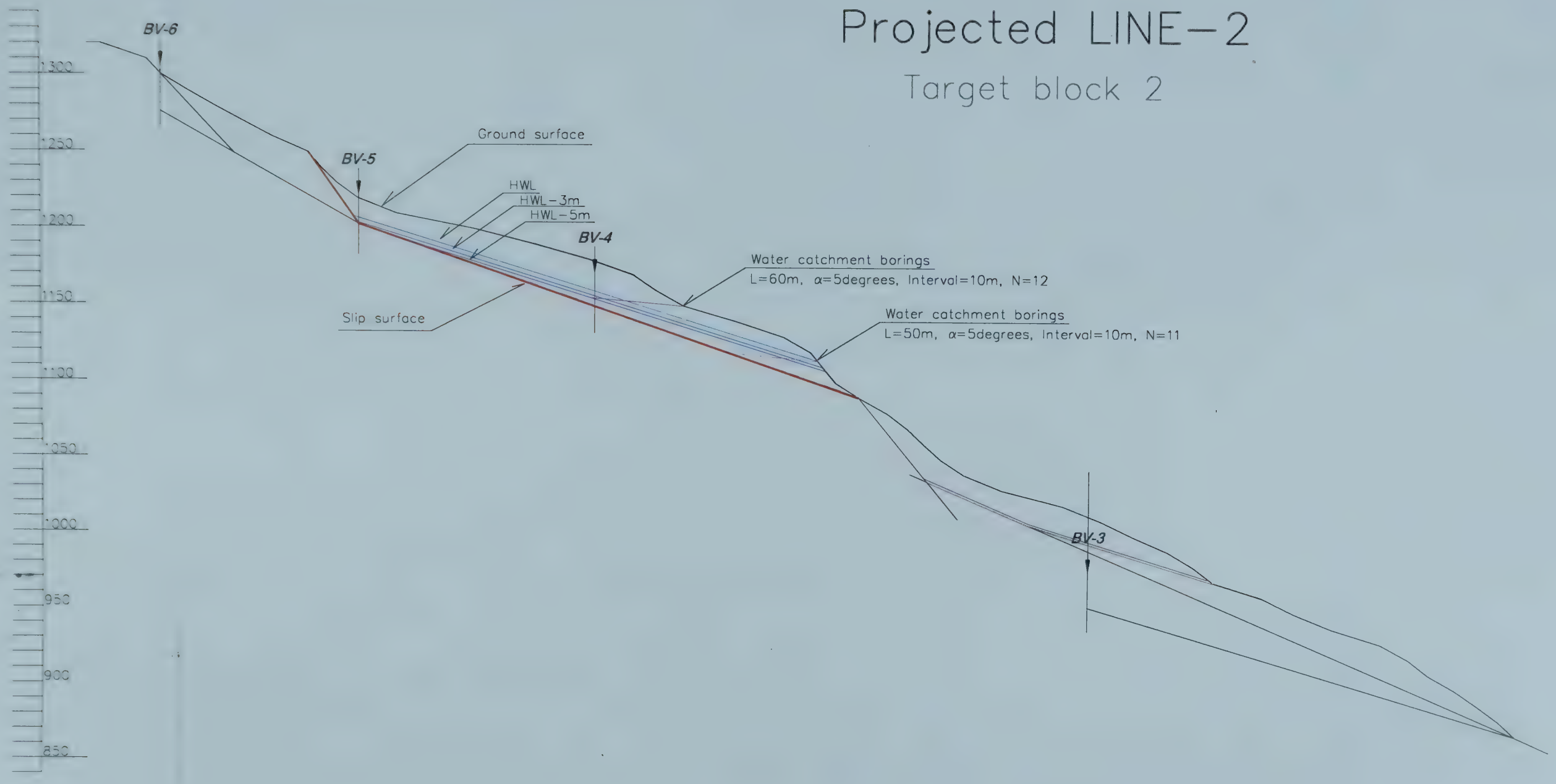
LINE−1

25+670.00

Target block 1

Projected LINE−2

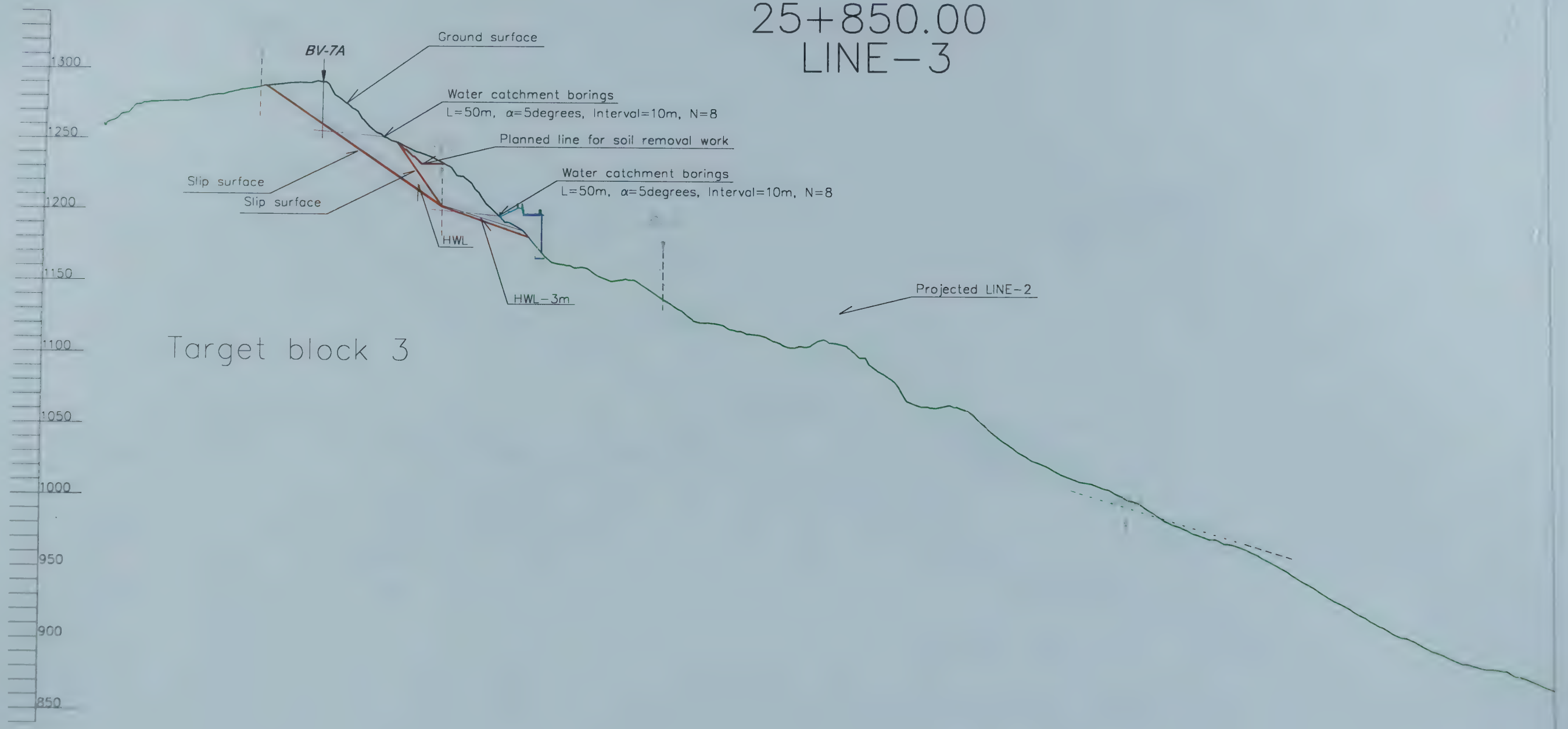
Target block 2



A04-51

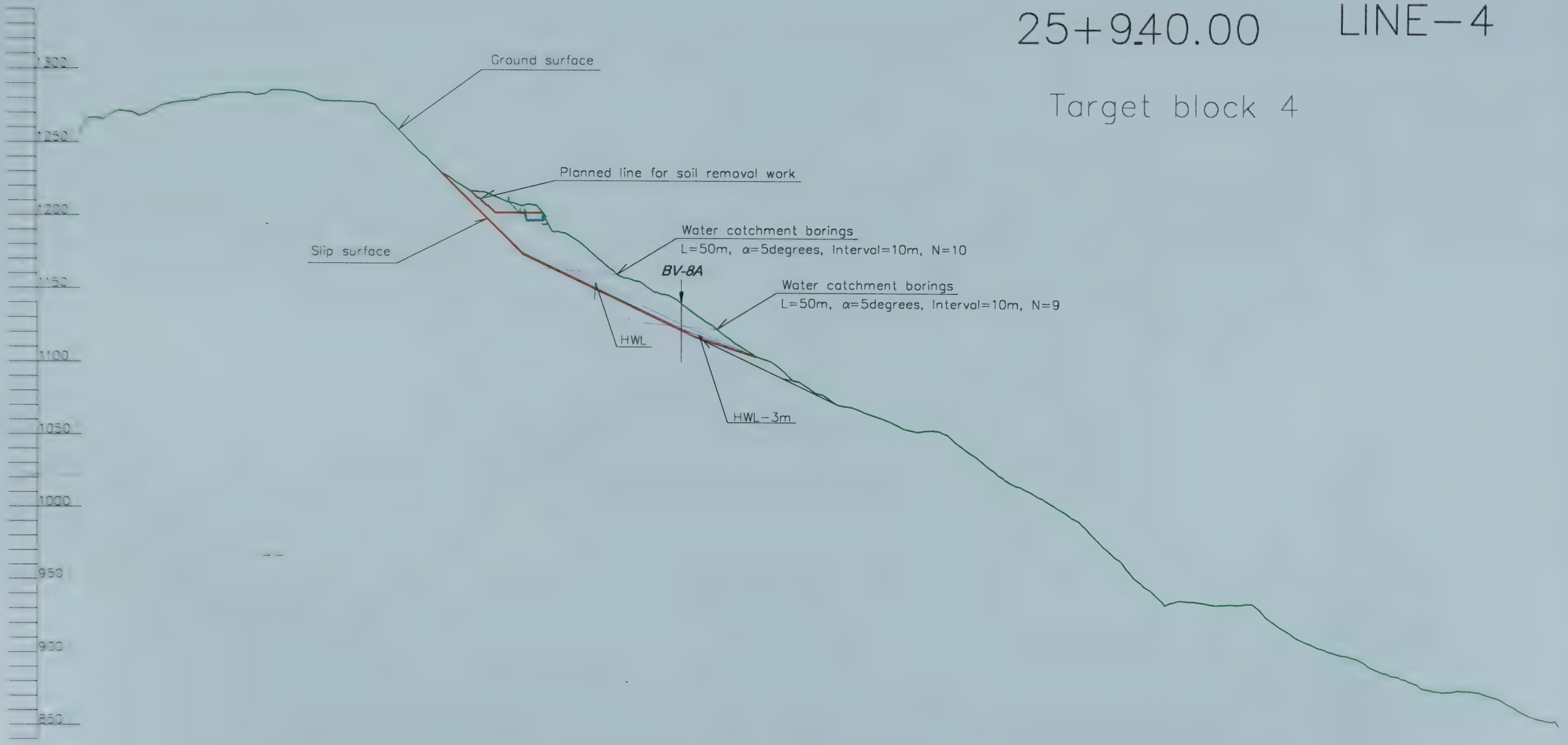
A04-52

25+850.00
LINE−3



25+940.00 LINE−4

Target block 4

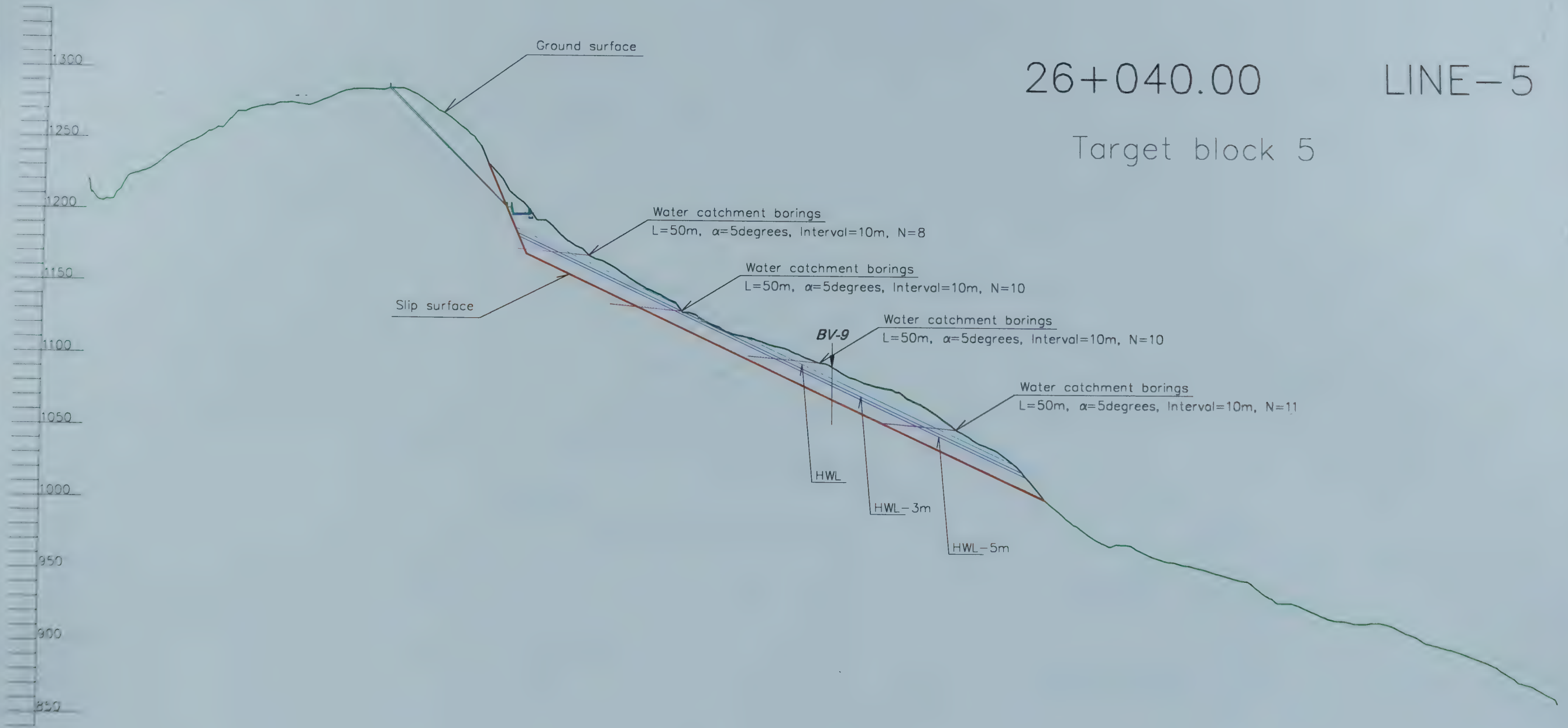


A04-53

A04-54

26+040.00 LINE-5

Target block 5

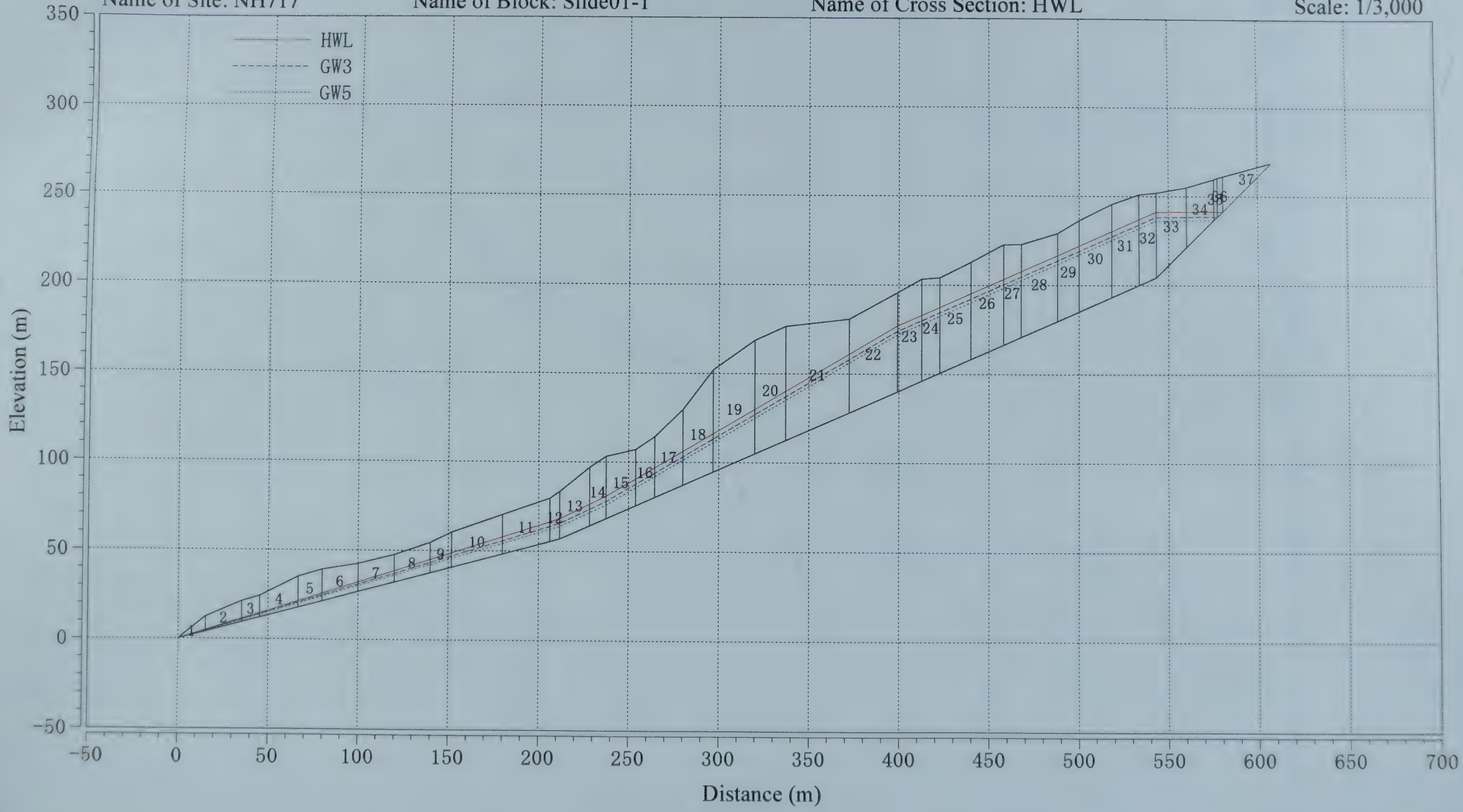


Results of slope stability analysis

Figures and Tables with English Translation

Cross Section

Name of Site: NH717 Name of Block: Slide01-1 Name of Cross Section: HWL Scale: 1/3,000



A04-56

Correlation Diagram of C'- ϕ'

Name of Site: NH717

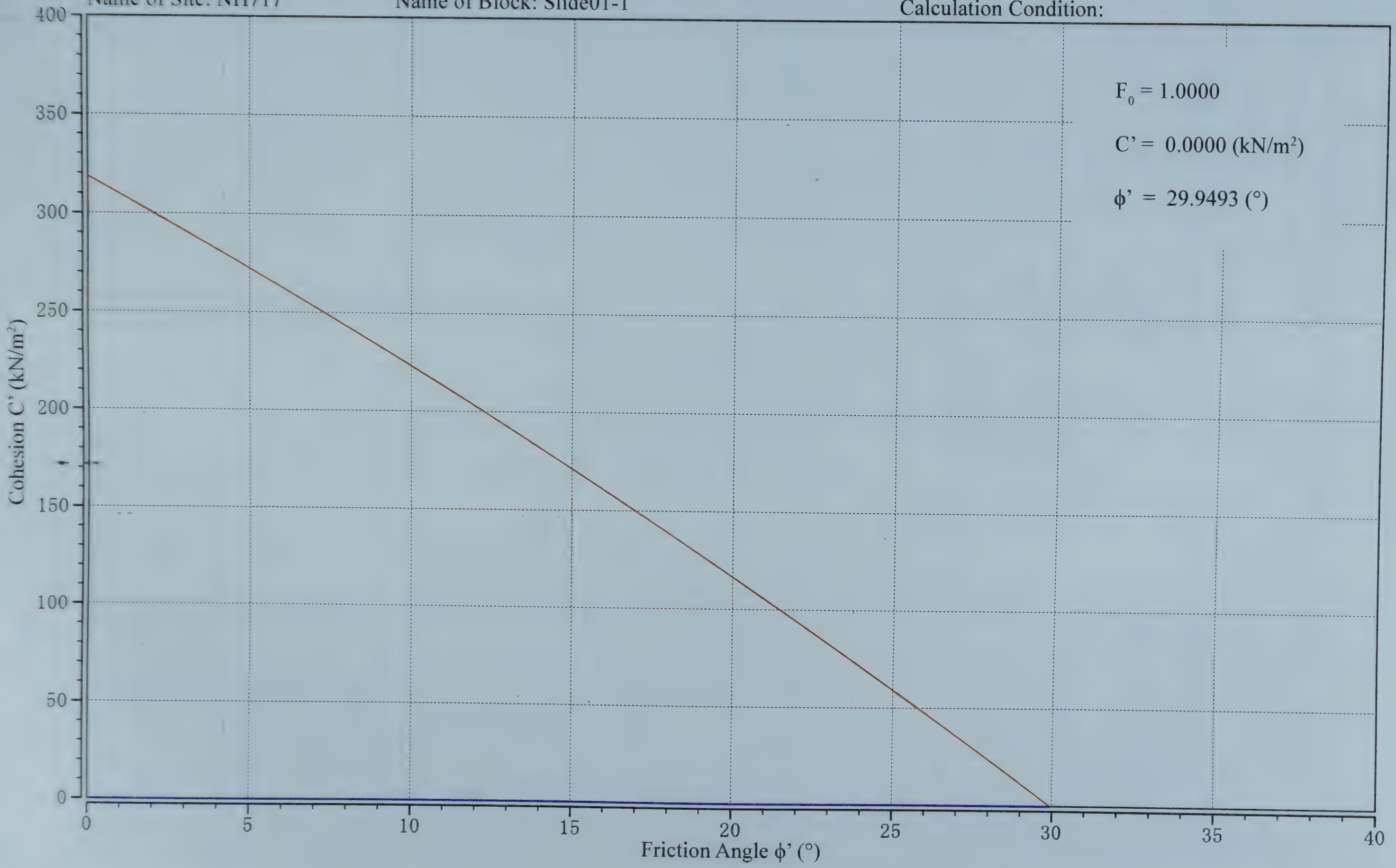
Name of Block: Slide01-1

Calculation Condition:

F_0 = 1.0000

C' = 0.0000 (kN/m²)

ϕ' = 29.9493 (°)



A04-57

Slidel whole dr ainage & remova

Spreadsheet for Two-dimensional Stability Analysis

Site	NH717	Block	Slide01-1	Analysis Method	Simplified Janbu Method	Scarplet at Downslope Side of Depression Zone	No.
Cross Section	HWL	Pore Wter Pressure	HWL	Groundwater Condition	Confined		
Calculation Condition				Horizontal Component of External Force Q (kN/m)	0.00	Horizontal Component of Seismic Intensity k_h	
				Reservoir Water	None	Elevation of Reference Water Surface BL (m)	
Geotechnical Parameters		Mean Value		Coordinate of Reference Point	x= , y= , z=		

Wet Unit Weight of Sliding Mass	γ (kN/m^3)	25.000	Designed Bearing Force	Numerator	Groundanchors P_{sa} (kN/m)
Submerged Unit Weight of Sliding Mass	γ' (kN/m^3)				Piles P_{sp} (kN/m)
Saturated Unit Weight of Sliding Mass	γ_{sat} (kN/m^3)				Others P_{sx} (kN/m)
Friction Angle of Depression Zone	ϕ_e (°)			Denominator	Groundanchors P_{ma} (kN/m)
Cohesion of Slip Surface	c' (kN/m^2)	0.0000			Piles P_{mp} (kN/m)
Friction Angle of Slip Surface	ϕ' (°)	29.9493			Others P_{mx} (kN/m)
Maximum Depth of Arc or Slip Surface	d (m)	33.78	Piezometric Height of Groundwater of Depression Zone		Z_w (m)
Length of Chord of Arc or Slip Surface	L (m)	664.42	Horizontal componet of water presssure		Q_w (kN/m)
Correction Factor	f_0	1.0282	Depth of Slip Surface at Downslope Side of Depression Zone		Z_h (m)
Inclination of Main Scarp	θ_u (°)		Coefficient of Active Earth Pressure		k_a
Inclination of Scarplet at Downslope Side of	θ_d (°)		Active Earth Pressure		E_a (kN/m)
Area of Sliding Mass	A (m^2)	21669.97	Two-dimensional Factors of Safety		F_2 1.0000
Shear Force of Landslide	Σ S (kN/m)	239971.29	$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$		
Driving Force of Landslide	Σ T (kN/m)	246726.34			

Slice	Depth of Slip Surface at Downslope Side h_1 (m)	Depth of Slip Surface at Upslope Side h_2 (m)	Horizontal Width of Slice b (m)	Inclination of Slip Surface α(°)	Weight W (kN/m)	Slice	Depth of Slip Surface at Downslope Side h_1 (m)	Depth of Slip Surface at Upslope Side h_2 (m)	Horizontal Width of Slice b (m)	Inclination of Slip Surface α(°)	Weight W (kN/m)
1	0.000	8.270	15.10	15.000	1560.96	36	21.900	19.620	3.02	45.000	1567.38
2	8.270	11.740	20.13	15.000	5035.02	37	19.620	0.000	27.24	45.000	6680.61
3	11.740	12.020	10.06	15.000	2987.82						
4	12.020	17.250	21.38	15.000	7822.41						
5	17.250	17.680	13.33	15.000	5820.21						
6	17.680	15.700	20.00	15.000	8345.00						
7	15.700	15.350	20.00	15.000	7762.50						
8	15.350	16.950	20.00	15.000	8075.00						
9	16.950	19.590	11.77	15.000	5375.95						
10	19.590	21.900	28.23	15.000	14640.78						
11	21.900	24.090	26.31	15.000	15124.96						
12	24.090	26.510	5.49	15.000	3472.43						
13	26.510	32.830	16.62	24.000	12327.89						
14	32.830	35.000	9.11	24.000	7724.14						
15	35.000	31.650	16.36	24.000	13629.93						
16	31.650	34.150	10.50	24.000	8636.25						
17	34.150	42.630	15.62	24.000	14991.30						
18	42.630	57.400	16.88	24.000	21106.33						
19	57.400	63.810	23.12	24.000	35029.69						
20	63.810	64.070	17.13	24.000	27382.31						
21	64.070	52.520	35.22	24.000	51328.75						
22	52.520	55.500	26.78	24.000	36159.70						
23	55.500	57.060	13.48	24.000	18966.36						
24	57.060	53.390	10.06	24.000	13889.09						
25	53.390	54.350	17.32	24.000	23325.71						
26	54.350	56.170	18.12	24.000	25032.78						
27	56.170	51.910	9.84	24.000	13293.84						
28	51.910	49.100	20.13	24.000	25416.64						
29	49.100	51.120	11.91	24.000	14920.25						
30	51.120	52.240	18.28	24.000	23617.76						
31	52.240	51.000	15.10	24.000	19486.55						
32	51.000	47.580	9.75	24.000	12014.44						
33	47.580	33.820	16.87	45.000	17165.23						
34	33.820	22.980	15.36	45.000	10905.60						
35	22.980	21.900	2.01	45.000	1127.61						

Note:

Slide1_whole_drainage & remova

Spreadsheet for Two-dimensional Stability Analysis

Site	NH717	Block	Slide01-1	Analysis Method	Simplified Janbu Method	Scarplet at Downslope Side	No.
Cross Section	HWL	Pore Wter Pressure	HWL	Groundwater Condition	Confined		
Calculation Condition				Horizontal Component of Reservoir Water	0.00	Horizontal Component of Elevation of Reference Water Surface	None
Geotechnical Parameters		Mean Value		Coordinate of Reference Point (m)	x= , y= , z=		
Wet Unit Weight of Sliding Mass	γ (kN/m^3)	25.000		Designed Bearing Force — Numerator — Groundanchors	P_{sa} (kN/m)		
Submerged Unit Weight of Sliding Mass	γ' (kN/m^3)			Piles	P_{sp} (kN/m)		
Saturated Unit Weight of Sliding Mass	γ_{sat} (kN/m^3)			Others	P_{sx} (kN/m)		
Friction Angle of Depression Zone	ϕ_s (°)			Denominator — Groundanchors	P_{ma} (kN/m)		
Cohesion of Slip Surface	c' (kN/m^2)	0.0000		Piles	P_{mp} (kN/m)		
Friction Angle of Slip Surface	ϕ' (°)	29.9493		Others	P_{mx} (kN/m)		
Shear Force of Landslide	Σ S (kN/m)	239971.29		Area of Sliding Mass	A (m^2)	21669.97	
Driving Force of Landslide	Σ T (kN/m)	246726.34		Two-dimensional Factors of Safety	F_2	1.0000	

Slice	Piezometric Height above Bottom of Slice U_2 (m)	Horizontal Width of Slice b_n (m)	Component Force in Normal Direction of Slip Surface N (kN/m)	Inclination of Seepage Pressure θ_p (°)	Seepage Pressure P (kN/m)	Pore Wter Pressure U (kN/m)	Length of Slip Surface L (m)	Inclination of Interslice Force from Horizontal α_i (°)	Vertical Shear Force Δx (kN/m)	Shear Force of Landslide s_1 (kN/m)	n_α	$S = s_1 / n_\alpha$ (kN/m)	Driving Force of Landslide T (kN/m)
1	0.810	15.10				60.0	15.63			864.83	1.0771	802.95	418.26
2	1.880	20.13				265.5	20.84			2748.05	1.0771	2551.45	1349.13
3	2.420	10.06				212.1	10.41			1599.28	1.0771	1484.87	800.58
4	3.560	21.38				626.9	22.13			4145.84	1.0771	3849.24	2096.01
5	4.280	13.33				512.4	13.80			3058.19	1.0771	2839.40	1559.52
6	5.350	20.00				944.4	20.71			4264.02	1.0771	3958.96	2236.04
7	6.420	20.00				1154.2	20.71			3807.49	1.0771	3535.09	2079.96
8	7.490	20.00				1364.1	20.71			3866.62	1.0771	3589.99	2163.69
9	8.110	11.77				900.3	12.19			2578.73	1.0771	2394.24	1440.48
10	9.620	28.23				2454.2	29.23			7021.56	1.0771	6519.21	3922.99
11	11.030	26.31				2664.0	27.24			7179.65	1.0771	6666.00	4052.72
12	11.320	5.49				601.6	5.68			1654.06	1.0771	1535.72	930.43
13	12.210	16.62				1917.5	18.19			5998.14	1.0487	5719.84	5488.73
14	12.700	9.11				1112.7	9.97			3809.31	1.0487	3632.57	3439.01
15	15.160	16.36				2234.9	17.91			6565.49	1.0487	6260.87	6068.43
16	16.740	10.50				1642.4	11.49			4029.67	1.0487	3842.71	3845.11
17	19.090	15.62				2744.2	17.10			7056.41	1.0487	6729.01	6674.55
18	21.620	16.88				3369.5	18.48			10219.45	1.0487	9745.30	9397.14
19	25.100	23.12				5296.4	25.31			17131.46	1.0487	16336.61	15596.22
20	27.680	17.13				4433.2	18.75			13222.61	1.0487	12609.12	12191.39
21	32.970	35.22				10474.0	38.55			23539.35	1.0487	22447.19	22853.03
22	37.000	26.78				9187.8	29.31			15540.41	1.0487	14819.37	16099.33
23	37.000	13.48				4891.2	14.76			8109.72	1.0487	7733.45	8444.37
24	37.000	10.06				3650.2	11.01			5899.33	1.0487	5625.62	6183.82
25	37.000	17.32				6284.5	18.96			9818.66	1.0487	9363.10	10385.28
26	37.000	18.12				6574.8	19.83			10634.97	1.0487	10141.54	11145.31
27	37.000	9.84				3570.4	10.77			5602.36	1.0487	5342.43	5918.80
28	37.000	20.13				7304.1	22.04			10435.93	1.0487	9951.73	11316.22
29	37.000	11.91				4321.5	13.04			6106.70	1.0487	5823.36	6642.92
30	37.000	18.28				6632.8	20.01			9786.23	1.0487	9332.17	10515.30
31	37.000	15.10				5479.0	16.53			8070.76	1.0487	7696.30	8675.97
32	37.000	9.75				3537.7	10.67			4884.02	1.0487	4657.42	5349.17
33	20.250	16.87				4735.7	23.86			7161.55	0.7881	9087.28	17165.23
34	5.000	15.36				1901.7	21.72			5187.78	0.7881	6582.77	10905.60
35	3.000	2.01				78.8	2.84			604.27	0.7881	766.75	1127.61
36	0.000	3.02				44.4	4.27			877.48	0.7881	1113.44	1567.38
37						0.0	38.52			3849.17	0.7881	4884.21	6680.61

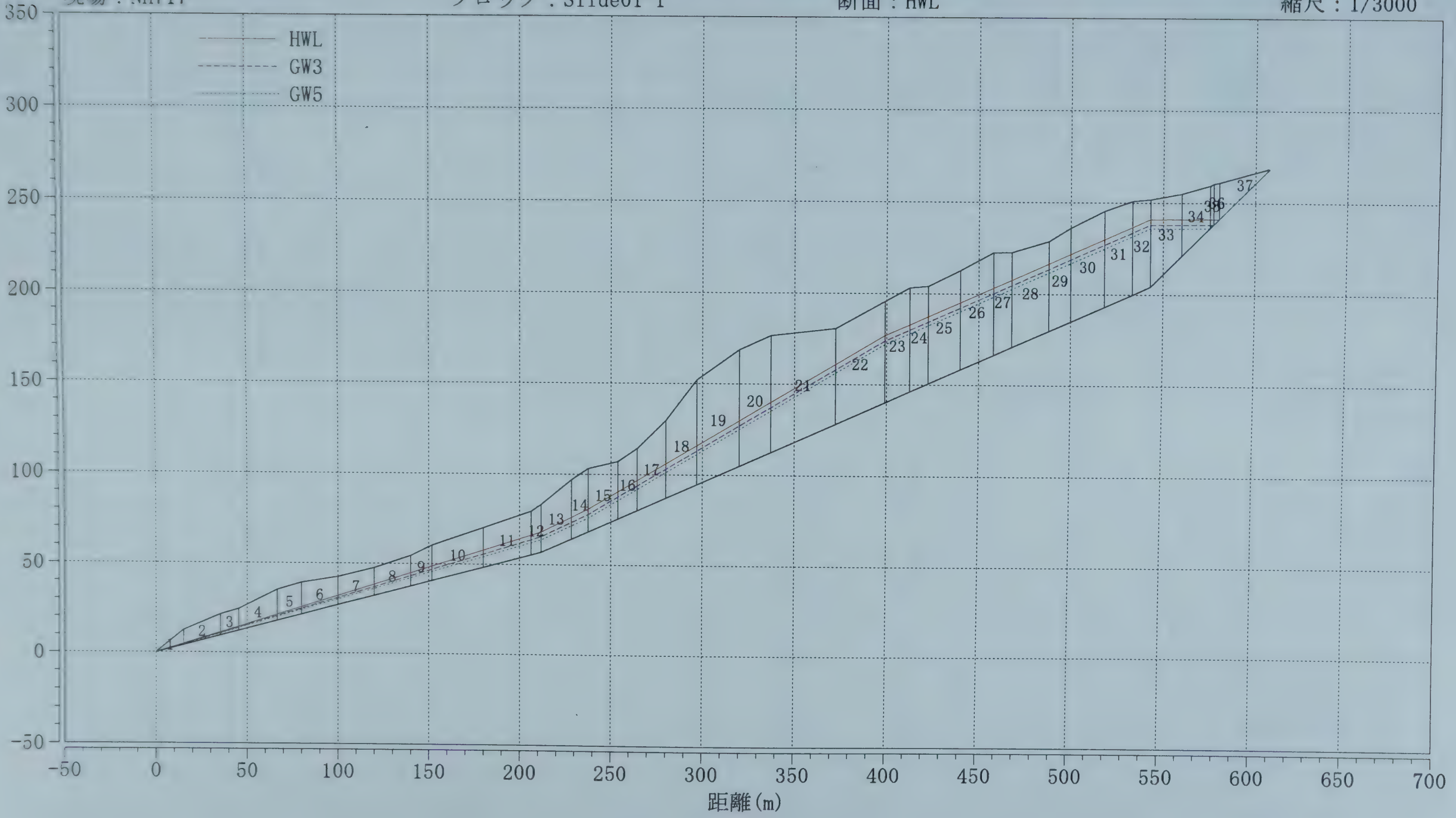
Note

Results of slope stability analysis

Line-1

縦断面図

現場：NH717　　ブロック：Slide01-1　　断面：HWL　　縮尺：1/3000



A04-61

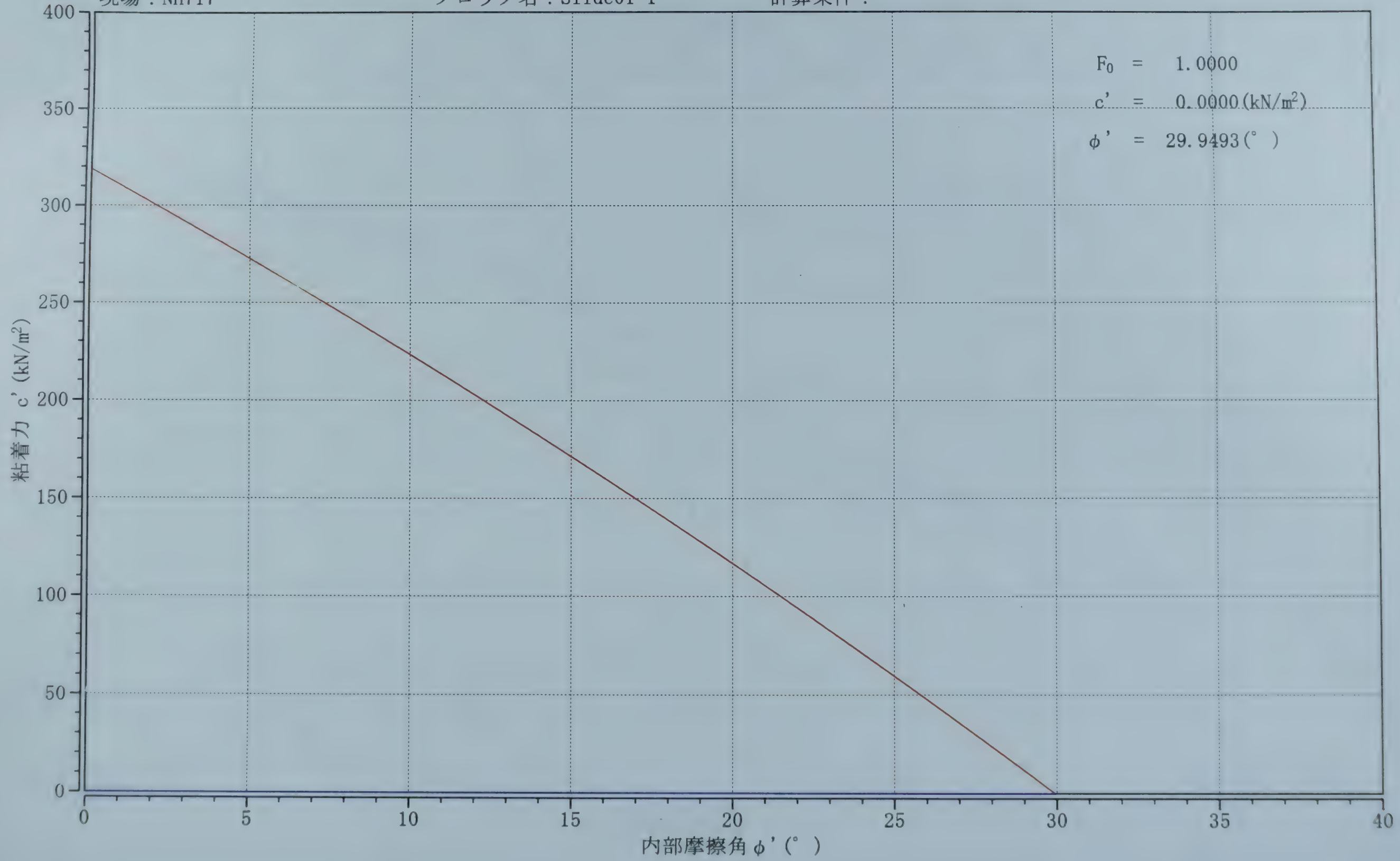
c’－φ’図

現場：NH717　　ブロック名：Slide01-1　　計算条件：

F_0 = 1.0000

c’ = 0.0000(kN/m²)

φ’ = 29.9493(°)



A04-62

Slide1_whole_drainage & remova

二　次　元　安　定　解　析　表

現場名	NH717	ブロック名	Slide01-1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x=　　, y=　　, z=		

移動層の湿潤単位体積重量	γ	(kN/m³)	25.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ'	(kN/m³)				杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat}	(kN/m³)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e	(°)			分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c'	(kN/m²)	0.0000			杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ'	(°)	29.9493			その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d	(m)	33.78	クラックの水位高		Z_w (m)	
地すべり斜面弦長	L	(m)	664.42	水圧の水平成分		Q_w (kN/m)	
修正係数	f_0		1.0282	陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u	(°)		主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d	(°)		主働土圧		E_a (kN/m)	
移動層の断面積	A	(m²)	21669.97	二次元安全率		F_2	1.0000
地すべり抵抗力	ΣS	(kN/m)	239971.29	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	ΣT	(kN/m)	246726.34				

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	8.270	15.10	15.000	1560.96	36	21.900	19.620	3.02	45.000	1567.38
2	8.270	11.740	20.13	15.000	5035.02	37	19.620	0.000	27.24	45.000	6680.61
3	11.740	12.020	10.06	15.000	2987.82						
4	12.020	17.250	21.38	15.000	7822.41						
5	17.250	17.680	13.33	15.000	5820.21						
6	17.680	15.700	20.00	15.000	8345.00						
7	15.700	15.350	20.00	15.000	7762.50						
8	15.350	16.950	20.00	15.000	8075.00						
9	16.950	19.590	11.77	15.000	5375.95						
10	19.590	21.900	28.23	15.000	14640.78						
11	21.900	24.090	26.31	15.000	15124.96						
12	24.090	26.510	5.49	15.000	3472.43						
13	26.510	32.830	16.62	24.000	12327.89						
14	32.830	35.000	9.11	24.000	7724.14						
15	35.000	31.650	16.36	24.000	13629.93						
16	31.650	34.150	10.50	24.000	8636.25						
17	34.150	42.630	15.62	24.000	14991.30						
18	42.630	57.400	16.88	24.000	21106.33						
19	57.400	63.810	23.12	24.000	35029.69						
20	63.810	64.070	17.13	24.000	27382.31						
21	64.070	52.520	35.22	24.000	51328.75						
22	52.520	55.500	26.78	24.000	36159.70						
23	55.500	57.060	13.48	24.000	18966.36						
24	57.060	53.390	10.06	24.000	13889.09						
25	53.390	54.350	17.32	24.000	23325.71						
26	54.350	56.170	18.12	24.000	25032.78						
27	56.170	51.910	9.84	24.000	13293.84						
28	51.910	49.100	20.13	24.000	25416.64						
29	49.100	51.120	11.91	24.000	14920.25						
30	51.120	52.240	18.28	24.000	23617.76						
31	52.240	51.000	15.10	24.000	19486.55						
32	51.000	47.580	9.75	24.000	12014.44						
33	47.580	33.820	16.87	45.000	17165.23						
34	33.820	22.980	15.36	45.000	10905.60						
35	22.980	21.900	2.01	45.000	1127.61						

備考：

Slide1_whole_drainage & removal

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide01-1	安定解析式	簡易Janbu法	陥没帯下流壁	なし
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
				末端座標 (m)	x= , y= , z		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	25.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	29.9493
地すべり抵抗力	Σ S (kN/m)	239971.29
地すべり力	Σ T (kN/m)	246726.34

設計済み負担推力			
分子側	アンカー	P_{sa} (kN/m)	
	杭	P_{sp} (kN/m)	
	その他	P_{sx} (kN/m)	
分母側	アンカー	P_{ma} (kN/m)	
	杭	P_{mp} (kN/m)	
	その他	P_{mx} (kN/m)	

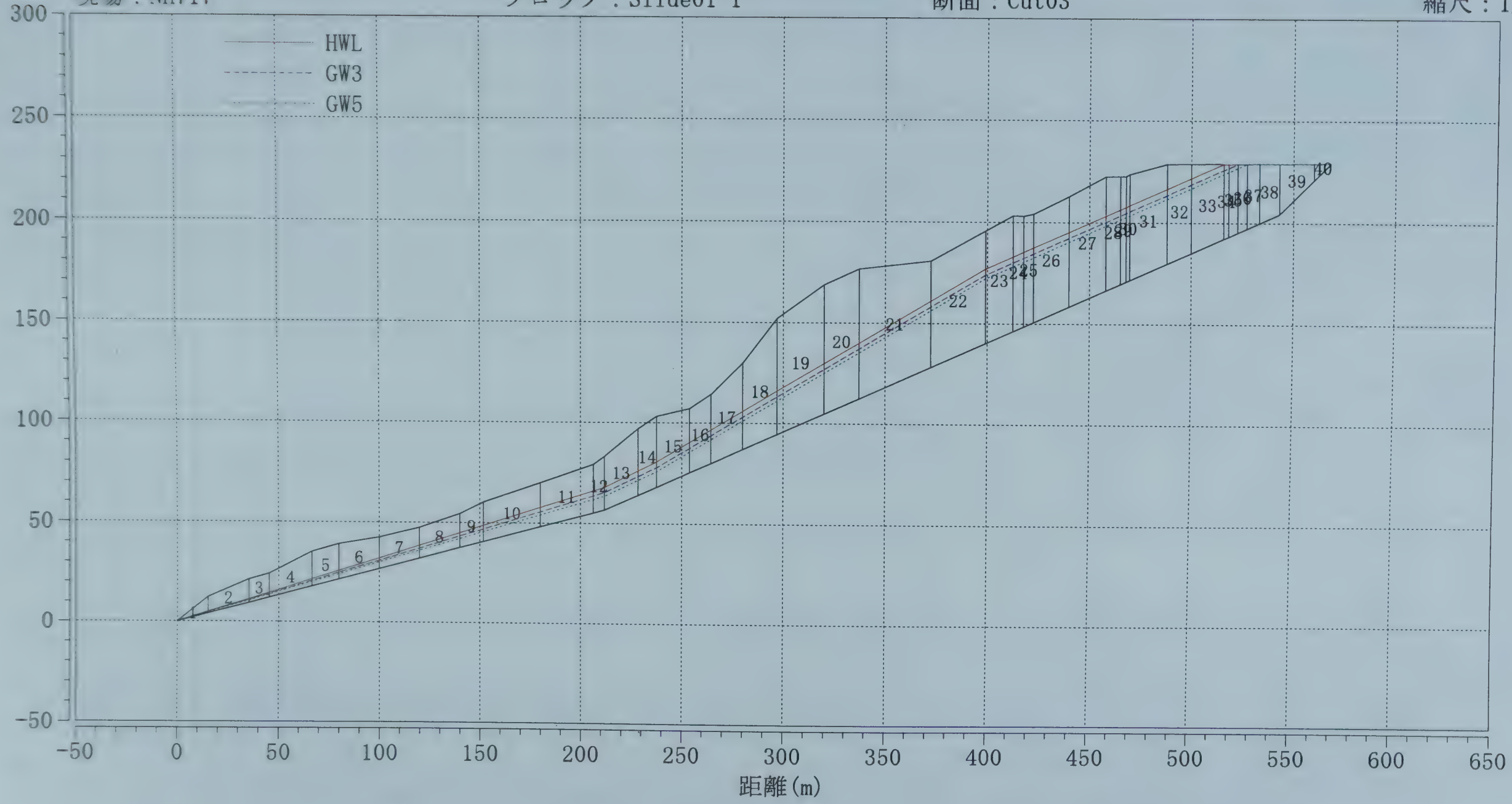
移動層の断面積	A (m^2)	21669.97
二次元安全率	F_2	1.0000

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	0.810	15.10				60.0	15.63			864.83	1.0771	802.95	418.26
2	1.880	20.13				265.5	20.84			2748.05	1.0771	2551.45	1349.13
3	2.420	10.06				212.1	10.41			1599.28	1.0771	1484.87	800.58
4	3.560	21.38				626.9	22.13			4145.84	1.0771	3849.24	2096.01
5	4.280	13.33				512.4	13.80			3058.19	1.0771	2839.40	1559.52
6	5.350	20.00				944.4	20.71			4264.02	1.0771	3958.96	2236.04
7	6.420	20.00				1154.2	20.71			3807.49	1.0771	3535.09	2079.96
8	7.490	20.00				1364.1	20.71			3866.62	1.0771	3589.99	2163.69
9	8.110	11.77				900.3	12.19			2578.73	1.0771	2394.24	1440.48
10	9.620	28.23				2454.2	29.23			7021.56	1.0771	6519.21	3922.99
11	11.030	26.31				2664.0	27.24			7179.65	1.0771	6666.00	4052.72
12	11.320	5.49				601.6	5.68			1654.06	1.0771	1535.72	930.43
13	12.210	16.62				1917.5	18.19			5998.14	1.0487	5719.84	5488.73
14	12.700	9.11				1112.7	9.97			3809.31	1.0487	3632.57	3439.01
15	15.160	16.36				2234.9	17.91			6565.49	1.0487	6260.87	6068.43
16	16.740	10.50				1642.4	11.49			4029.67	1.0487	3842.71	3845.11
17	19.090	15.62				2744.2	17.10			7056.41	1.0487	6729.01	6674.55
18	21.620	16.88				3369.5	18.48			10219.45	1.0487	9745.30	9397.14
19	25.100	23.12				5296.4	25.31			17131.46	1.0487	16336.61	15596.22
20	27.680	17.13				4433.2	18.75			13222.61	1.0487	12609.12	12191.39
21	32.970	35.22				10474.0	38.55			23539.35	1.0487	22447.19	22853.03
22	37.000	26.78				9187.8	29.31			15540.41	1.0487	14819.37	16099.33
23	37.000	13.48				4891.2	14.76			8109.72	1.0487	7733.45	8444.37
24	37.000	10.06				3650.2	11.01			5899.33	1.0487	5625.62	6183.82
25	37.000	17.32				6284.5	18.96			9818.66	1.0487	9363.10	10385.28
26	37.000	18.12				6574.8	19.83			10634.97	1.0487	10141.54	11145.31
27	37.000	9.84				3570.4	10.77			5602.36	1.0487	5342.43	5918.80
28	37.000	20.13				7304.1	22.04			10435.93	1.0487	9951.73	11316.22
29	37.000	11.91				4321.5	13.04			6106.70	1.0487	5823.36	6642.92
30	37.000	18.28				6632.8	20.01			9786.23	1.0487	9332.17	10515.30
31	37.000	15.10				5479.0	16.53			8070.76	1.0487	7696.30	8675.97
32	37.000	9.75				3537.7	10.67			4884.02	1.0487	4657.42	5349.17
33	20.250	16.87				4735.7	23.86			7161.55	0.7881	9087.28	17165.23
34	5.000	15.36				1901.7	21.72			5187.78	0.7881	6582.77	10905.60
35	3.000	2.01				78.8	2.84			604.27	0.7881	766.75	1127.61
36	0.000	3.02				44.4	4.27			877.48	0.7881	1113.44	1567.38
37						0.0	38.52			3849.17	0.7881	4884.21	6680.61

備考：

縦断面図

現場：NH717　　ブロック：Slide01-1　　断面：Cut03　　縮尺：1/3000



A04-63

Slide1_whole_drainage & removal

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide01-1	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut03	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m^3)	25.000	設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)				杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)				その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)			分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000			杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	29.9493			その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	26.69	クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	613.01	水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0234	陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧		E_a (kN/m)	
移動層の断面積	A (m^2)	19732.56	二次元安全率		F_2	1.1113
地すべり抵抗力	Σ S (kN/m)	226717.03	$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	Σ T (kN/m)	208791.13				

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	8.270	15.10	15.000	1560.96	36	34.000	32.000	4.59	24.000	3786.75
2	8.270	11.740	20.13	15.000	5035.02	37	32.000	29.360	6.06	24.000	4648.02
3	11.740	12.020	10.06	15.000	2987.82	38	29.360	25.110	9.75	24.000	6638.53
4	12.020	17.250	21.38	15.000	7822.41	39	25.110	8.360	16.87	45.000	7057.99
5	17.250	17.680	13.33	15.000	5820.21	40	8.360	0.000	8.42	45.000	879.89
6	17.680	15.700	20.00	15.000	8345.00						
7	15.700	15.350	20.00	15.000	7762.50						
8	15.350	16.950	20.00	15.000	8075.00						
9	16.950	19.590	11.77	15.000	5375.95						
10	19.590	21.900	28.23	15.000	14640.78						
11	21.900	24.090	26.31	15.000	15124.96						
12	24.090	26.510	5.49	15.000	3472.43						
13	26.510	32.830	16.62	24.000	12327.89						
14	32.830	35.000	9.11	24.000	7724.14						
15	35.000	31.650	16.36	24.000	13629.93						
16	31.650	34.150	10.50	24.000	8636.25						
17	34.150	42.630	15.62	24.000	14991.30						
18	42.630	57.400	16.88	24.000	21106.33						
19	57.400	63.810	23.12	24.000	35029.69						
20	63.810	64.070	17.13	24.000	27382.31						
21	64.070	52.520	35.22	24.000	51328.75						
22	52.520	55.500	26.78	24.000	36159.70						
23	55.500	57.060	13.48	24.000	18966.36						
24	57.060	54.630	5.19	24.000	7245.89						
25	54.630	53.390	4.87	24.000	6575.72						
26	53.390	54.350	17.32	24.000	23325.71						
27	54.350	56.170	18.12	24.000	25032.78						
28	56.170	53.150	7.00	24.000	9565.50						
29	53.150	51.910	2.84	24.000	3729.63						
30	51.910	52.190	1.75	24.000	2277.19						
31	52.190	49.100	18.38	24.000	23271.38						
32	49.100	43.910	11.91	24.000	13846.86						
33	43.910	37.000	15.84	24.000	16020.18						
34	37.000	35.940	2.44	24.000	2224.67						
35	35.940	34.000	4.44	24.000	3881.67						

備考：

Slide1_whole_drainage & remova

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide01-1	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	Cut03	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	25.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m³)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	29.9493				その他 P_{mx} (kN/m)	
地すべり抵抗力	ΣS (kN/m)	226717.03		移動層の断面積	A (m²)	19732.56	
地すべり力	ΣT (kN/m)	208791.13		二次元安全率	F_2	1.1113	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_n (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	0.490	15.10				36.3	15.63			878.48	1.0626	826.70	418.26
2	1.140	20.13				160.9	20.84			2808.33	1.0626	2642.81	1349.13
3	1.470	10.06				128.7	10.41			1647.32	1.0626	1550.22	800.58
4	2.160	21.38				380.5	22.13			4287.79	1.0626	4035.07	2096.01
5	2.590	13.33				310.5	13.80			3174.56	1.0626	2987.45	1559.52
6	3.240	20.00				571.7	20.71			4478.73	1.0626	4214.76	2236.04
7	3.890	20.00				699.2	20.71			4069.66	1.0626	3829.80	2079.96
8	4.540	20.00				826.7	20.71			4176.26	1.0626	3930.11	2163.69
9	4.920	11.77				546.0	12.19			2782.90	1.0626	2618.88	1440.48
10	5.840	28.23				1489.4	29.23			7577.44	1.0626	7130.83	3922.99
11	6.690	26.31				1616.5	27.24			7783.21	1.0626	7324.47	4052.72
12	6.870	5.49				365.0	5.68			1790.39	1.0626	1684.87	930.43
13	7.400	16.62				1162.9	18.19			6432.94	1.0272	6262.50	5488.73
14	7.700	9.11				674.5	9.97			4061.80	1.0272	3954.18	3439.01
15	10.160	16.36				1432.7	17.91			7027.69	1.0272	6841.49	6068.43
16	11.740	10.50				1127.5	11.49			4326.31	1.0272	4211.69	3845.11
17	14.090	15.62				1978.3	17.10			7497.70	1.0272	7299.05	6674.55
18	16.620	16.88				2541.8	18.48			10696.34	1.0272	10412.95	9397.14
19	20.100	23.12				4162.8	25.31			17784.63	1.0272	17313.44	15596.22
20	22.680	17.13				3593.3	18.75			13706.56	1.0272	13343.41	12191.39
21	27.970	35.22				8747.0	38.55			24534.37	1.0272	23884.35	22853.03
22	32.000	26.78				7874.7	29.31			16296.98	1.0272	15865.21	16099.33
23	32.000	13.48				4230.2	14.76			8490.55	1.0272	8265.60	8444.37
24	32.000	5.19				1628.7	5.68			3236.47	1.0272	3150.72	3226.08
25	32.000	4.87				1528.3	5.33			2908.19	1.0272	2831.14	2927.70
26	32.000	17.32				5435.2	18.96			10307.97	1.0272	10034.87	10385.28
27	32.000	18.12				5686.3	19.83			11146.89	1.0272	10851.56	11145.31
28	32.000	7.00				2196.7	7.66			4245.70	1.0272	4133.21	4258.83
29	32.000	2.84				891.2	3.11			1635.40	1.0272	1592.08	1660.54
30	32.000	1.75				549.2	1.92			995.63	1.0272	969.25	1013.87
31	32.000	18.38				5767.9	20.12			10085.01	1.0272	9817.81	10361.08
32	32.000	11.91				3737.5	13.04			5824.72	1.0272	5670.40	6165.02
33	32.000	15.84				4970.8	17.34			6366.34	1.0272	6197.66	7132.64
34	32.000	2.44				765.7	2.67			840.61	1.0272	818.34	990.49
35	32.000	4.44				1393.3	4.86			1433.71	1.0272	1395.73	1728.23
36	32.000	4.59				1440.4	5.02			1351.90	1.0272	1316.08	1685.97
37	29.360	6.06				1823.3	6.63			1627.54	1.0272	1584.42	2069.43
38	25.110	9.75				2604.1	10.67			2324.54	1.0272	2262.95	2955.66
39	8.360	16.87				2768.6	23.86			2471.42	0.7592	3255.13	7057.99
40	0.000	8.42				345.2	11.91			308.10	0.7592	405.80	879.89

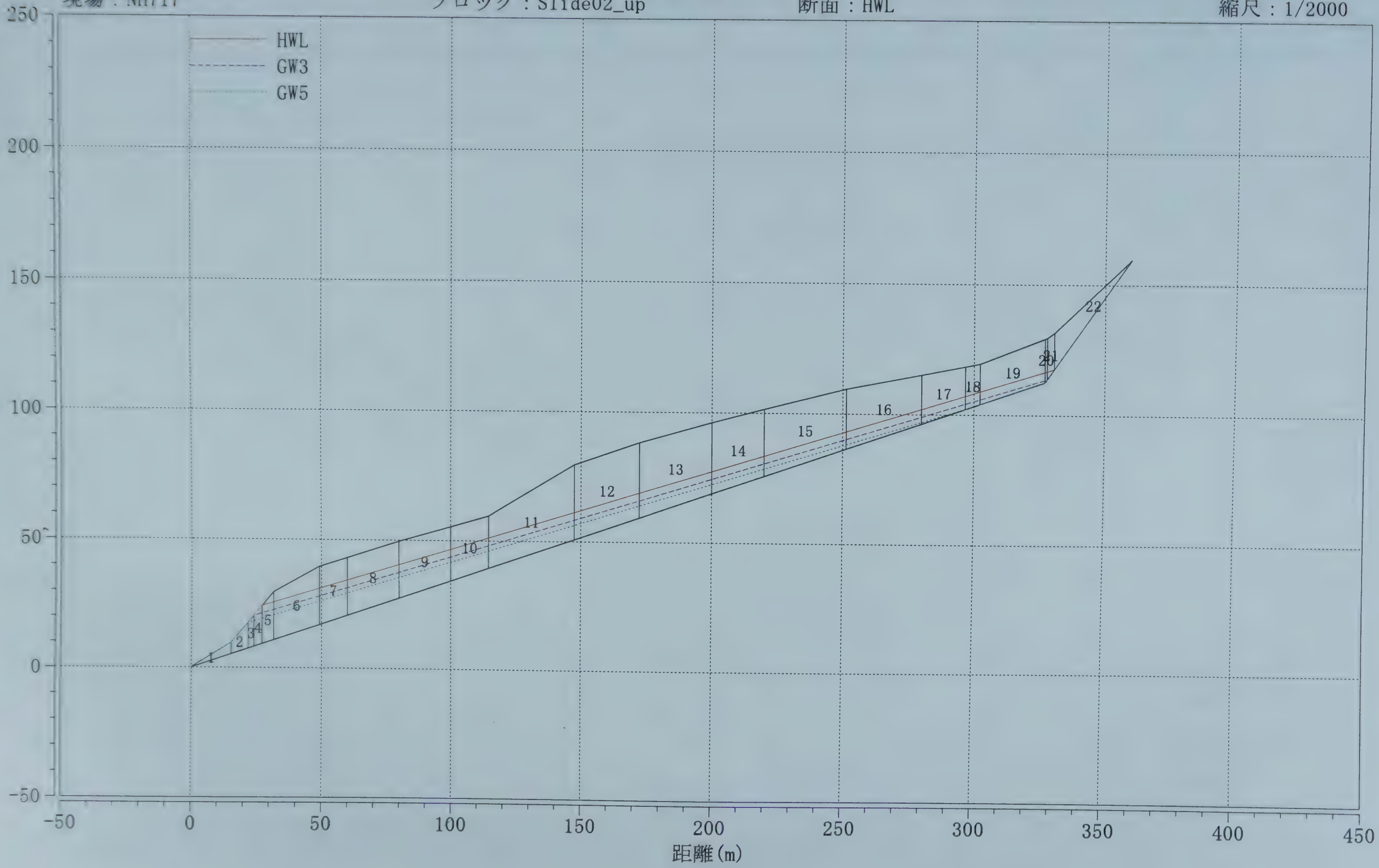
備考

Results of slope stability analysis

Line-2

縦断面図

現場：NH717　　ブロック：Slide02_up　　断面：HWL　　縮尺：1/2000



A04-69

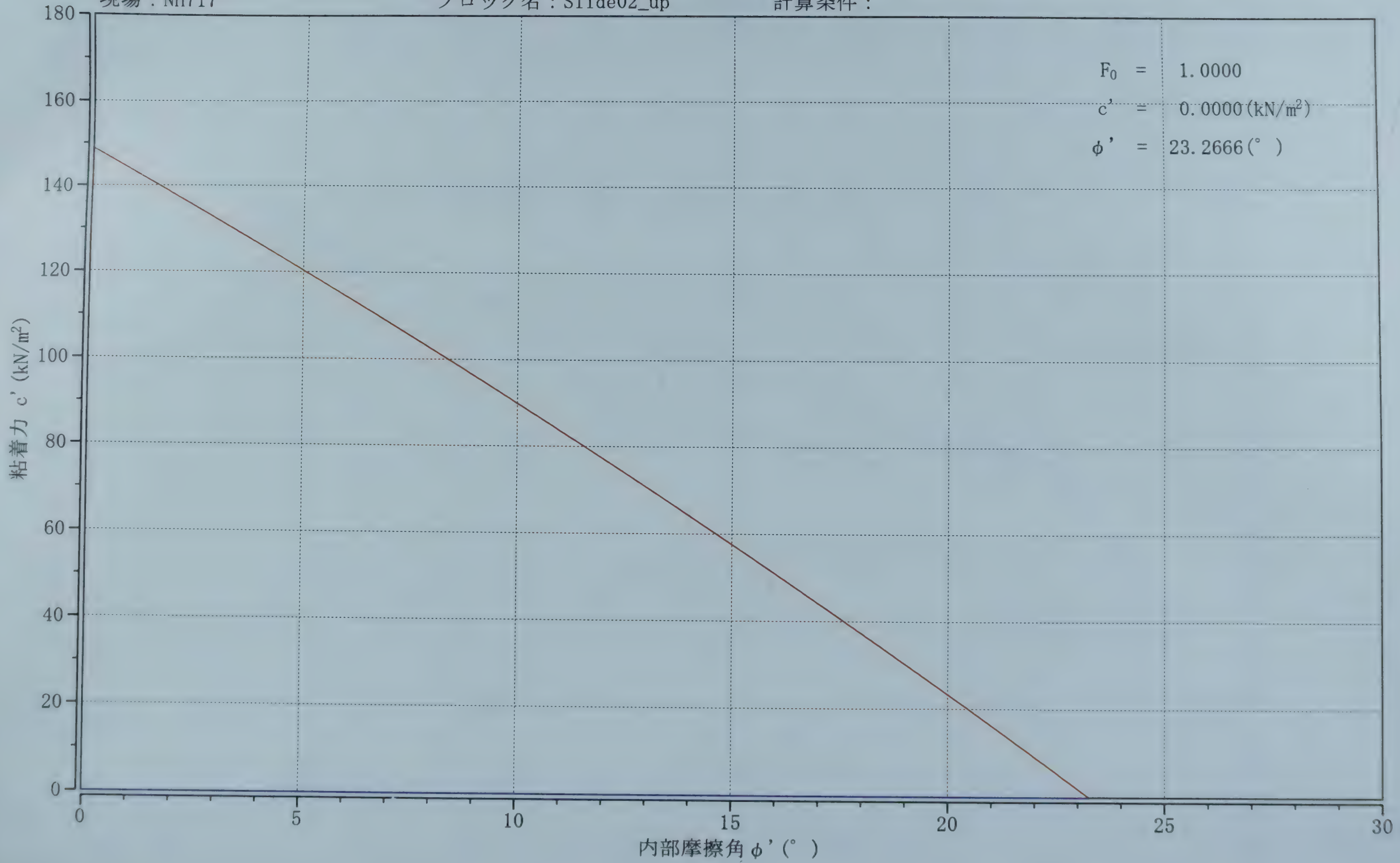
c’－φ’図

現場：NH717　　ブロック名：Slide02_up　　計算条件：

F_0 = 1.0000

c’ = 0.0000(kN/m²)

φ’ = 23.2666(°)



A04-70

Slide2_upward_drainage work.WS

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide02_up	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター	平均値			末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m^3)	26.000	設計済み負担推力 分子側	アンカー P_{sa} (kN/m)
移動層の水中単位体積重量	γ' (kN/m^3)			杭 P_{sp} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)			その他 P_{sx} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)		分母側	アンカー P_{ma} (kN/m)
すべり面材料の粘着力	c' (kN/m^2)	0.0000		杭 P_{mp} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	23.2666		その他 P_{mx} (kN/m)
弦～すべり面の最大深さ	d (m)	29.63	クラックの水位高	Z_w (m)
地すべり斜面弦長	L (m)	393.85	水圧の水平成分	Q_w (kN/m)
修正係数	f_0	1.0402	陥没帯下流壁のすべり面深さ	Z_h (m)
冠頭部クラックの傾斜角	θ_u (°)		主働土圧係数	k_a
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧	E_a (kN/m)
移動層の断面積	A (m^2)	7238.20	二次元安全率	F_2 1.0000
地すべり抵抗力	ΣS (kN/m)	69322.66		
地すべり力	ΣT (kN/m)	72111.27		

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	4.340	15.17	19.000	855.89	15	25.830	23.030	31.30	19.000	19881.13
2	4.340	9.950	6.48	19.000	1203.79	16	23.030	18.780	28.70	19.000	15599.31
3	9.950	11.870	2.21	19.000	626.89	17	18.780	16.310	16.69	19.000	7613.48
4	11.870	14.760	3.25	19.000	1125.12	18	16.310	15.480	5.59	19.000	2310.18
5	14.760	18.700	4.45	19.000	1935.66	19	15.480	16.660	24.81	19.000	10366.11
6	18.700	22.600	17.73	19.000	9519.24	20	16.660	15.870	0.81	55.000	342.54
7	22.600	22.410	10.73	19.000	6278.44	21	15.870	14.010	2.69	55.000	1044.90
8	22.410	22.060	20.00	19.000	11562.20	22	14.010	0.000	29.42	55.000	5358.26
9	22.060	20.930	20.00	19.000	11177.40						
10	20.930	20.100	14.60	19.000	7787.49						
11	20.100	28.790	32.86	19.000	20884.83						
12	28.790	28.910	24.88	19.000	18662.49						
13	28.910	27.340	27.67	19.000	20233.69						
14	27.340	25.830	20.00	19.000	13824.20						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_l / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	4.340	15.17				322.8	16.04			229.21	1.0264	223.32	294.71
2	9.950	6.48				454.0	6.85			322.37	1.0264	314.09	414.50
3	11.870	2.21				236.4	2.34			167.88	1.0264	163.57	215.86
4	14.760	3.25				424.4	3.44			301.31	1.0264	293.57	387.41
5	14.590	4.45				640.4	4.71			556.93	1.0264	542.62	666.50
6	13.950	17.73				2481.2	18.75			3026.22	1.0264	2948.48	3277.74
7	13.570	10.73				1447.9	11.35			2077.02	1.0264	2023.67	2161.84
8	12.840	20.00				2589.9	21.15			3857.87	1.0264	3758.77	3981.18
9	12.120	20.00				2447.7	21.15			3753.55	1.0264	3657.13	3848.69
10	11.590	14.60				1697.4	15.44			2618.62	1.0264	2551.35	2681.45
11	10.400	32.86				3543.1	34.75			7456.55	1.0264	7265.00	7191.22
12	9.500	24.88				2427.7	26.31			6980.59	1.0264	6801.27	6426.01
13	8.500	27.67				2442.2	29.26			7649.95	1.0264	7453.44	6967.02
14	7.780	20.00				1596.5	21.15			5257.62	1.0264	5122.56	4760.05
15	6.640	31.30				2213.1	33.10			7596.85	1.0264	7401.70	6845.62
16	5.600	28.70				1722.5	30.35			5966.72	1.0264	5813.45	5371.27
17	5.000	16.69				867.5	17.65			2900.63	1.0264	2826.12	2621.53
18	4.800	5.59				268.6	5.91			877.83	1.0264	855.28	795.46
19	3.900	24.81				1058.4	26.24			4002.12	1.0264	3899.31	3569.34
20	3.000	0.81				27.4	1.41			135.50	0.5310	255.18	489.20
21	0.000	2.69				39.6	4.69			432.27	0.5310	814.05	1492.28
22						0.0	51.29			2303.93	0.5310	4338.75	7652.39

備考

Slide2_upward_drainage work.WS

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide02_up	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ	(kN/m^3)	26.000	
移動層の水中単位体積重量	γ'	(kN/m^3)		
移動層の飽和単位体積重量	γ_{sat}	(kN/m^3)		
陥没帯の内部摩擦角	ϕ_e	(°)		
すべり面材料の粘着力	c'	(kN/m^2)	0.0000	
すべり面材料の内部摩擦角	ϕ'	(°)	23.2666	
弦～すべり面の最大深さ	d	(m)	29.63	
地すべり斜面弦長	L	(m)	393.85	
修正係数	f_0		1.0402	
冠頭部クラックの傾斜角	θ_u	(°)		
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d	(°)		
移動層の断面積	A	(m^2)	7238.20	
地すべり抵抗力	Σ S	(kN/m)	76600.74	
地すべり力	Σ T	(kN/m)	72111.27	

設計済み負担推力				
分子側	アンカー	P_{sa}	(kN/m)	
	杭	P_{sp}	(kN/m)	
	その他	P_{sx}	(kN/m)	
分母側	アンカー	P_{ma}	(kN/m)	
	杭	P_{mp}	(kN/m)	
	その他	P_{mx}	(kN/m)	

クラックの水位高	Z_w	(m)	
水圧の水平成分	Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ	Z_h	(m)	
主働土圧係数	k_a		
主働土圧	E_a	(kN/m)	
二次元安全率	F_2		1.1050

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	4.340	15.17	19.000	855.89	15	25.830	23.030	31.30	19.000	19881.13
2	4.340	9.950	6.48	19.000	1203.79	16	23.030	18.780	28.70	19.000	15599.31
3	9.950	11.870	2.21	19.000	626.89	17	18.780	16.310	16.69	19.000	7613.48
4	11.870	14.760	3.25	19.000	1125.12	18	16.310	15.480	5.59	19.000	2310.18
5	14.760	18.700	4.45	19.000	1935.66	19	15.480	16.660	24.81	19.000	10366.11
6	18.700	22.600	17.73	19.000	9519.24	20	16.660	15.870	0.81	55.000	342.54
7	22.600	22.410	10.73	19.000	6278.44	21	15.870	14.010	2.69	55.000	1044.90
8	22.410	22.060	20.00	19.000	11562.20	22	14.010	0.000	29.42	55.000	5358.26
9	22.060	20.930	20.00	19.000	11177.40						
10	20.930	20.100	14.60	19.000	7787.49						
11	20.100	28.790	32.86	19.000	20884.83						
12	28.790	28.910	24.88	19.000	18662.49						
13	28.910	27.340	27.67	19.000	20233.69						
14	27.340	25.830	20.00	19.000	13824.20						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	4.340	15.17				322.8	16.04			229.21	1.0138	226.09	294.71
2	9.950	6.48				454.0	6.85			322.37	1.0138	317.99	414.50
3	9.870	2.21				214.8	2.34			177.20	1.0138	174.79	215.86
4	9.760	3.25				312.8	3.44			349.27	1.0138	344.52	387.41
5	9.590	4.45				422.2	4.71			650.75	1.0138	641.90	666.50
6	8.950	17.73				1611.8	18.75			3400.02	1.0138	3353.77	3277.74
7	8.570	10.73				921.8	11.35			2303.25	1.0138	2271.91	2161.84
8	7.840	20.00				1609.3	21.15			4279.53	1.0138	4221.32	3981.18
9	7.120	20.00				1467.1	21.15			4175.22	1.0138	4118.42	3848.69
10	6.590	14.60				981.5	15.44			2926.43	1.0138	2886.62	2681.45
11	5.400	32.86				1931.9	34.75			8149.34	1.0138	8038.48	7191.22
12	4.500	24.88				1207.7	26.31			7505.14	1.0138	7403.05	6426.01
13	3.500	27.67				1085.4	29.26			8233.33	1.0138	8121.33	6967.02
14	2.780	20.00				615.9	21.15			5679.29	1.0138	5602.03	4760.05
15	1.640	31.30				678.4	33.10			8256.76	1.0138	8144.44	6845.62
16	0.600	28.70				315.2	30.35			6571.81	1.0138	6482.41	5371.27
17	0.000	16.69				49.1	17.65			3252.51	1.0138	3208.26	2621.53
18						0.0	5.91			993.32	1.0138	979.81	795.46
19						0.0	26.24			4457.19	1.0138	4396.56	3569.34
20						0.0	1.41			147.28	0.5118	287.77	489.20
21						0.0	4.69			449.28	0.5118	877.82	1492.28
22						0.0	51.29			2303.93	0.5118	4501.45	7652.39

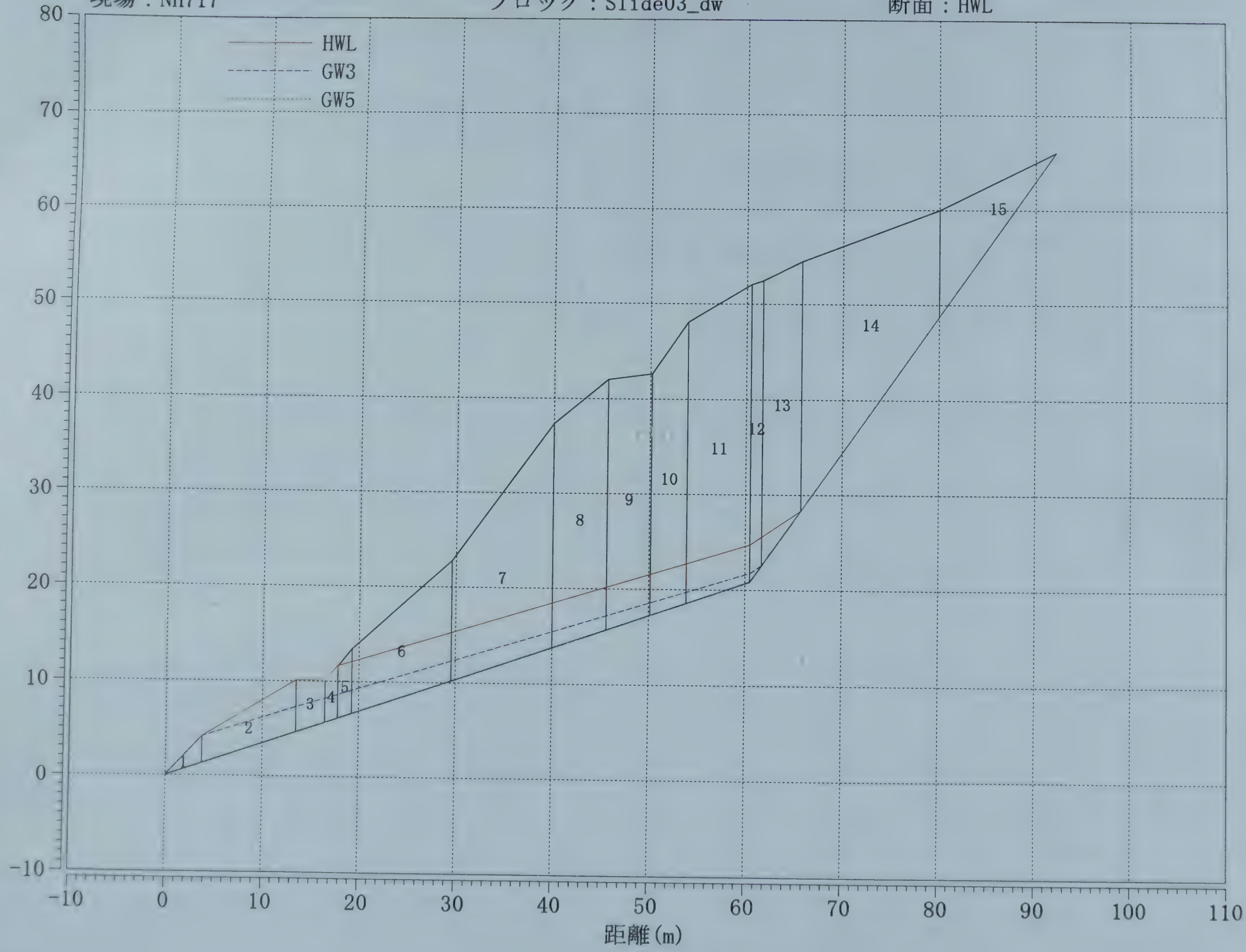
備考：

Results of slope stability analysis

Line-3 Down

縦断面図

現場：NH717　　ブロック：Slide03_dw　　断面：HWL　　縮尺：1/600



A04-74

c’－φ’図

現場：NH717　　ブロック名：Slide03_dw　　計算条件：

F_0 = 1.0000

c’ = 0.0000(kN/m²)

φ’ = 31.7529(°)

粘着力 c’(kN/m²)

180
160
140
120
100
80
60
40
20
0

0　5　10　15　20　25　30　35　40

内部摩擦角 φ’(°)

A04-75

Slide3 downward _drainage & rem

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide03_dw	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
				末端座標 (m)	x= , y= , z=		

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	26.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	31.7529
弦～すべり面の最大深さ	d (m)	18.31
地すべり斜面弦長	L (m)	113.38
修正係数	f_0	1.0641
冠頭部クラックの傾斜角	θ_u (°)	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)	
移動層の断面積	A (m^2)	1409.80
地すべり抵抗力	Σ S (kN/m)	24745.04
地すべり力	Σ T (kN/m)	26332.34

設計済み負担推力				
分子側	アンカー	P_{sa}	(kN/m)	
	杭	P_{sp}	(kN/m)	
	その他	P_{sx}	(kN/m)	
分母側	アンカー	P_{ma}	(kN/m)	
	杭	P_{mp}	(kN/m)	
	その他	P_{mx}	(kN/m)	

クラックの水位高	Z_w	(m)	
水圧の水平成分	Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ	Z_h	(m)	
主働土圧係数	k_a		
主働土圧	E_a	(kN/m)	
二次元安全率	F_2		1.0000

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

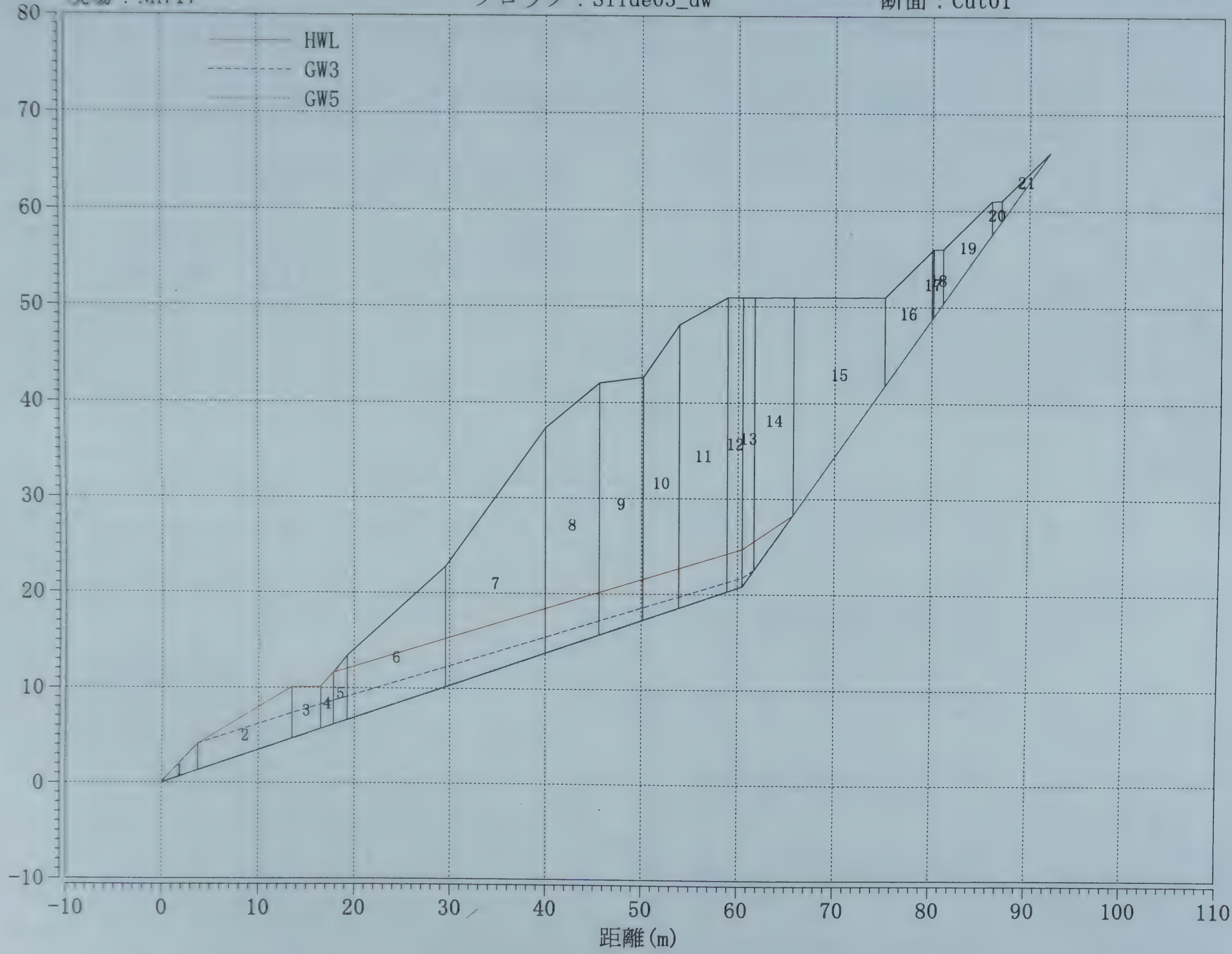
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	2.760	3.72	19.000	133.47	15	11.300	0.000	12.15	55.000	1784.84
2	2.760	5.390	9.78	19.000	1036.19						
3	5.390	4.400	3.00	19.000	381.81						
4	4.400	5.440	1.33	19.000	170.13						
5	5.440	6.780	1.44	19.000	228.76						
6	6.780	12.580	10.30	19.000	2592.30						
7	12.580	23.560	10.43	19.000	4900.22						
8	23.560	26.320	5.62	19.000	3644.23						
9	26.320	25.370	4.54	19.000	3050.74						
10	25.370	29.520	3.73	19.000	2661.62						
11	29.520	31.230	6.59	19.000	5204.45						
12	31.230	29.920	1.21	55.000	961.89						
13	29.920	26.210	4.03	55.000	2940.65						
14	26.210	11.300	14.28	55.000	6963.36						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_l / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	2.760	3.72				50.3	3.93			51.45	1.0845	47.44	45.96
2	5.390	9.78				390.8	10.34			399.41	1.0845	368.28	356.79
3	4.400	3.00				144.0	3.17			147.17	1.0845	135.70	131.47
4	5.440	1.33				64.2	1.41			65.58	1.0845	60.47	58.58
5	5.390	1.44				76.5	1.52			94.25	1.0845	86.91	78.77
6	5.020	10.30				525.7	10.89			1278.97	1.0845	1179.30	892.60
7	4.640	10.43				494.0	11.03			2726.94	1.0845	2514.43	1687.28
8	4.440	5.62				250.2	5.94			2100.52	1.0845	1936.82	1254.81
9	4.270	4.54				193.9	4.80			1768.07	1.0845	1630.28	1050.46
10	4.140	3.73				153.8	3.94			1552.05	1.0845	1431.10	916.47
11	3.900	6.59				259.8	6.97			3060.19	1.0845	2821.71	1792.04
12	3.000	1.21				40.9	2.11			569.97	0.6198	919.64	1373.72
13	0.000	4.03				59.3	7.03			1783.25	0.6198	2877.26	4199.68
14						0.0	24.90			4309.54	0.6198	6953.43	9944.70
15						0.0	21.18			1104.61	0.6198	1782.29	2549.01

備考：

縦断面図

現場：NH717　　ブロック：Slide03_dw　　断面：Cut01　　縮尺：1/600



A04-77

Slide3_downward
_drainage & rem

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide03_dw	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut01	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	26.000		設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)
移動層の水中単位体積重量	γ' (kN/m³)					杭	P_{sp} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m³)					その他	P_{sx} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー	P_{ma} (kN/m)
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭	P_{mp} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	31.7529				その他	P_{mx} (kN/m)
弦～すべり面の最大深さ	d (m)	18.31		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	113.38		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0641		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	1288.16		二次元安全率		F_2	1.1841
地すべり抵抗力	Σ S (kN/m)	24321.92		$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	Σ T (kN/m)	21857.99					

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	2.760	3.72	19.000	133.47	15	22.630	9.200	9.43	55.000	3902.04
2	2.760	5.390	9.78	19.000	1036.19	16	9.200	7.150	4.85	55.000	1030.87
3	5.390	4.400	3.00	19.000	381.81	17	7.150	7.080	0.15	55.000	27.75
4	4.400	5.440	1.33	19.000	170.13	18	7.080	5.660	1.00	55.000	165.62
5	5.440	6.780	1.44	19.000	228.76	19	5.660	3.540	5.00	55.000	598.00
6	6.780	12.580	10.30	19.000	2592.30	20	3.540	2.120	1.00	55.000	73.58
7	12.580	23.560	10.43	19.000	4900.22	21	2.120	0.000	5.00	55.000	137.80
8	23.560	26.320	5.62	19.000	3644.23						
9	26.320	25.370	4.54	19.000	3050.74						
10	25.370	29.520	3.73	19.000	2661.62						
11	29.520	30.640	5.01	19.000	3918.22						
12	30.640	30.090	1.58	19.000	1247.39						
13	30.090	28.370	1.21	55.000	919.58						
14	28.370	22.630	4.03	55.000	2671.89						

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_l / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1						0.0	3.93			82.61	1.0549	78.31	45.96
2						0.0	10.34			641.29	1.0549	607.91	356.79
3						0.0	3.17			236.30	1.0549	224.00	131.47
4						0.0	1.41			105.29	1.0549	99.81	58.58
5						0.0	1.52			141.58	1.0549	134.21	78.77
6						0.0	10.89			1604.35	1.0549	1520.85	892.60
7						0.0	11.03			3032.69	1.0549	2874.86	1687.28
8						0.0	5.94			2255.37	1.0549	2138.00	1254.81
9						0.0	4.80			1888.07	1.0549	1789.81	1050.46
10						0.0	3.94			1647.24	1.0549	1561.52	916.47
11						0.0	5.30			2424.94	1.0549	2298.74	1349.15
12						0.0	1.67			772.00	1.0549	731.82	429.51
13						0.0	2.11			569.12	0.5746	990.51	1313.29
14						0.0	7.03			1653.60	0.5746	2878.01	3815.85
15						0.0	16.44			2414.93	0.5746	4203.05	5572.69
16						0.0	8.46			637.99	0.5746	1110.39	1472.23
17						0.0	0.26			17.17	0.5746	29.89	39.63
18						0.0	1.74			102.50	0.5746	178.40	236.53
19						0.0	8.72			370.10	0.5746	644.13	854.03
20						0.0	1.74			45.54	0.5746	79.26	105.08
21						0.0	8.72			85.28	0.5746	148.43	196.80

備考：

Results of slope stability analysis

Line-3 Whole

縦断面図

現場：NH717　　ブロック：Slide03_wh　　断面：HWL　　縮尺：1/1000

HWL
GW3
GW5

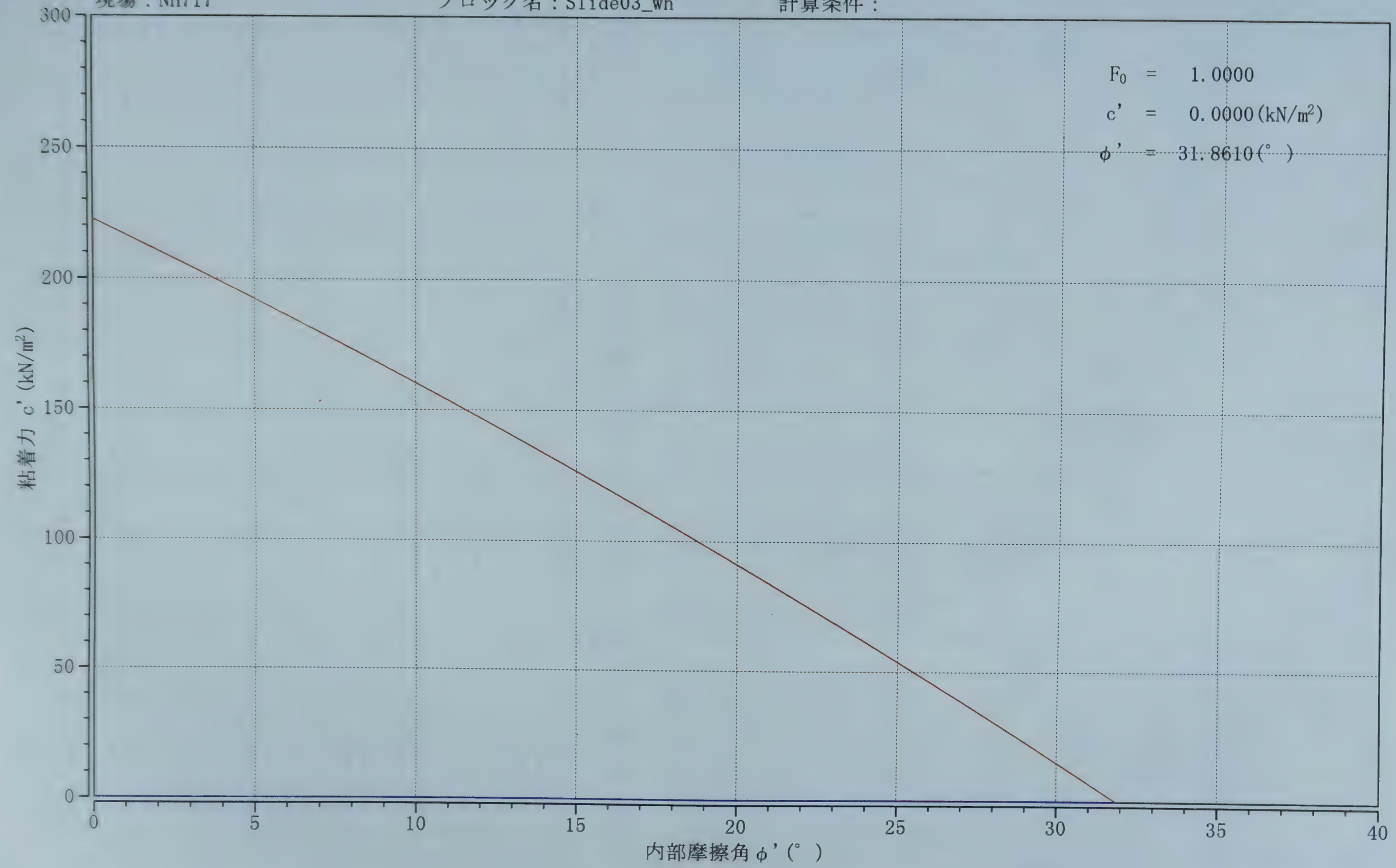
標高(m)

距離(m)

A04-80

c　－φ'　図

現場：NH717　　ブロック名：Slide03_wh　　計算条件：



A04-81

Slide3_whole_drainage work.WS1

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide03_wh	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	

土質パラメーター	平均値	末端座標 (m)	x= , y= , z=
移動層の湿潤単位体積重量	γ (kN/m³)	26.000	設計済み負担推力
移動層の水中単位体積重量	γ' (kN/m³)		分子側 アンカー P_{sa} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m³)		分子側 杭 P_{sp} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)		分子側 その他 P_{sx} (kN/m)
すべり面材料の粘着力	c' (kN/m²)	0.0000	分母側 アンカー P_{ma} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	31.8610	分母側 杭 P_{mp} (kN/m)
弦～すべり面の最大深さ	d (m)	11.70	分母側 その他 P_{mx} (kN/m)
地すべり斜面弦長	L (m)	212.91	クラックの水位高 Z_w (m)
修正係数	f_0	1.0305	水圧の水平成分 Q_w (kN/m)
冠頭部クラックの傾斜角	θ_u (°)		陥没帯下流壁のすべり面深さ Z_h (m)
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		主働土圧係数 k_a
移動層の断面積	A (m²)	3711.67	主働土圧 E_a (kN/m)
地すべり抵抗力	ΣS (kN/m)	55472.70	二次元安全率 F_2 1.0000
地すべり力	ΣT (kN/m)	57166.70	

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	2.760	3.72	19.000	133.47	15	25.820	23.560	12.15	34.000	7799.57
2	2.760	5.390	9.78	19.000	1036.19	16	23.560	21.470	7.85	34.000	4595.31
3	5.390	4.400	3.00	19.000	381.81	17	21.470	20.380	6.79	34.000	3694.10
4	4.400	5.440	1.33	19.000	170.13	18	20.380	26.040	13.21	34.000	7971.71
5	5.440	6.780	1.44	19.000	228.76	19	26.040	26.510	16.58	34.000	11326.63
6	6.780	12.580	10.30	19.000	2592.30	20	26.510	31.350	4.13	34.000	3106.50
7	12.580	23.560	10.43	19.000	4900.22	21	31.350	31.500	2.11	34.000	1723.98
8	23.560	26.320	5.62	19.000	3644.23	22	31.500	30.000	1.99	34.000	1591.01
9	26.320	25.370	4.54	19.000	3050.74	23	30.000	27.800	4.16	34.000	3125.82
10	25.370	29.520	3.73	19.000	2661.62	24	27.800	23.790	4.11	34.000	2756.45
11	29.520	31.230	6.59	19.000	5204.45	25	23.790	18.430	8.29	34.000	4550.05
12	31.230	30.820	1.21	34.000	976.05	26	18.430	3.900	18.82	34.000	5463.26
13	30.820	30.110	4.03	34.000	3192.12	27	3.900	0.900	3.65	34.000	227.76
14	30.110	25.820	14.28	34.000	10382.85	28	0.900	0.000	1.39	34.000	16.26

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	2.760	3.72				50.3	3.93			51.67	1.0853	47.60	45.96
2	5.390	9.78				390.8	10.34			401.09	1.0853	369.56	356.79
3	4.400	3.00				144.0	3.17			147.79	1.0853	136.17	131.47
4	5.440	1.33				64.2	1.41			65.86	1.0853	60.68	58.58
5	5.390	1.44				76.5	1.52			94.65	1.0853	87.21	78.77
6	5.020	10.30				525.7	10.89			1284.37	1.0853	1183.40	892.60
7	4.640	10.43				494.0	11.03			2738.45	1.0853	2523.17	1687.28
8	4.440	5.62				250.2	5.94			2109.39	1.0853	1943.56	1254.81
9	4.270	4.54				193.9	4.80			1775.54	1.0853	1635.95	1050.46
10	4.140	3.73				153.8	3.94			1558.60	1.0853	1436.07	916.47
11	3.900	6.59				259.8	6.97			3073.11	1.0853	2831.52	1792.04
12	3.900	1.21				46.3	1.46			577.85	0.9754	592.41	658.35
13	3.900	4.03				154.1	4.86			1888.12	0.9754	1935.68	2153.11
14	3.900	14.28				546.2	17.22			6113.52	0.9754	6267.53	7003.32
15	3.900	12.15				464.7	14.66			4558.64	0.9754	4673.48	5260.88
16	3.900	7.85				300.2	9.47			2669.40	0.9754	2736.65	3099.58
17	3.900	6.79				259.7	8.19			2134.49	0.9754	2188.26	2491.70
18	3.900	13.21				505.2	15.93			4640.42	0.9754	4757.33	5376.98
19	3.900	16.58				634.1	20.00			6645.41	0.9754	6812.82	7639.91
20	3.900	4.13				158.0	4.98			1832.53	0.9754	1878.69	2095.36
21	3.900	2.11				80.7	2.55			1021.30	0.9754	1047.03	1162.84
22	3.900	1.99				76.1	2.40			941.51	0.9754	965.23	1073.15
23	3.900	4.16				159.1	5.02			1843.82	0.9754	1890.27	2108.39
24	3.900	4.11				157.2	4.96			1615.44	0.9754	1656.14	1859.25
25	3.900	8.29				317.1	10.00			2630.81	0.9754	2697.09	3069.05
26	3.900	18.82				719.8	22.70			2948.07	0.9754	3022.34	3685.01
27	0.900	3.65				85.9	4.40			88.16	0.9754	90.38	153.63
28	0.000	1.39				6.1	1.68			6.30	0.9754	6.45	10.97

備考

Slide3_whole_drainage work.WS1

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide03_wh	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	

土質パラメーター		平均値
移動層の湿潤単位体積重量	γ (kN/m^3)	26.000
移動層の水中単位体積重量	γ' (kN/m^3)	
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)	
陥没帯の内部摩擦角	ϕ_e (°)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000
すべり面材料の内部摩擦角	ϕ' (°)	31.8610
弦～すべり面の最大深さ	d (m)	11.70
地すべり斜面弦長	L (m)	212.91
修正係数	f_0	1.0305
冠頭部クラックの傾斜角	θ_u (°)	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)	
移動層の断面積	A (m^2)	3711.67
地すべり抵抗力	Σ S (kN/m)	61577.26
地すべり力	Σ T (kN/m)	57166.70

設計済み負担推力				
分子側	アンカー	P_{sa} (kN/m)		
	杭	P_{sp} (kN/m)		
	その他	P_{sx} (kN/m)		
分母側	アンカー	P_{ma} (kN/m)		
	杭	P_{mp} (kN/m)		
	その他	P_{mx} (kN/m)		

クラックの水位高	Z_w (m)	
水圧の水平成分	Q_w (kN/m)	
陥没帯下流壁のすべり面深さ	Z_h (m)	
主働土圧係数	k_a	
主働土圧	E_a (kN/m)	
二次元安全率	F_2	1.1101

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	2.760	3.72	19.000	133.47	15	25.820	23.560	12.15	34.000	7799.57
2	2.760	5.390	9.78	19.000	1036.19	16	23.560	21.470	7.85	34.000	4595.31
3	5.390	4.400	3.00	19.000	381.81	17	21.470	20.380	6.79	34.000	3694.10
4	4.400	5.440	1.33	19.000	170.13	18	20.380	26.040	13.21	34.000	7971.71
5	5.440	6.780	1.44	19.000	228.76	19	26.040	26.510	16.58	34.000	11326.63
6	6.780	12.580	10.30	19.000	2592.30	20	26.510	31.350	4.13	34.000	3106.50
7	12.580	23.560	10.43	19.000	4900.22	21	31.350	31.500	2.11	34.000	1723.98
8	23.560	26.320	5.62	19.000	3644.23	22	31.500	30.000	1.99	34.000	1591.01
9	26.320	25.370	4.54	19.000	3050.74	23	30.000	27.800	4.16	34.000	3125.82
10	25.370	29.520	3.73	19.000	2661.62	24	27.800	23.790	4.11	34.000	2756.45
11	29.520	31.230	6.59	19.000	5204.45	25	23.790	18.430	8.29	34.000	4550.05
12	31.230	30.820	1.21	34.000	976.05	26	18.430	3.900	18.82	34.000	5463.26
13	30.820	30.110	4.03	34.000	3192.12	27	3.900	0.900	3.65	34.000	227.76
14	30.110	25.820	14.28	34.000	10382.85	28	0.900	0.000	1.39	34.000	16.26

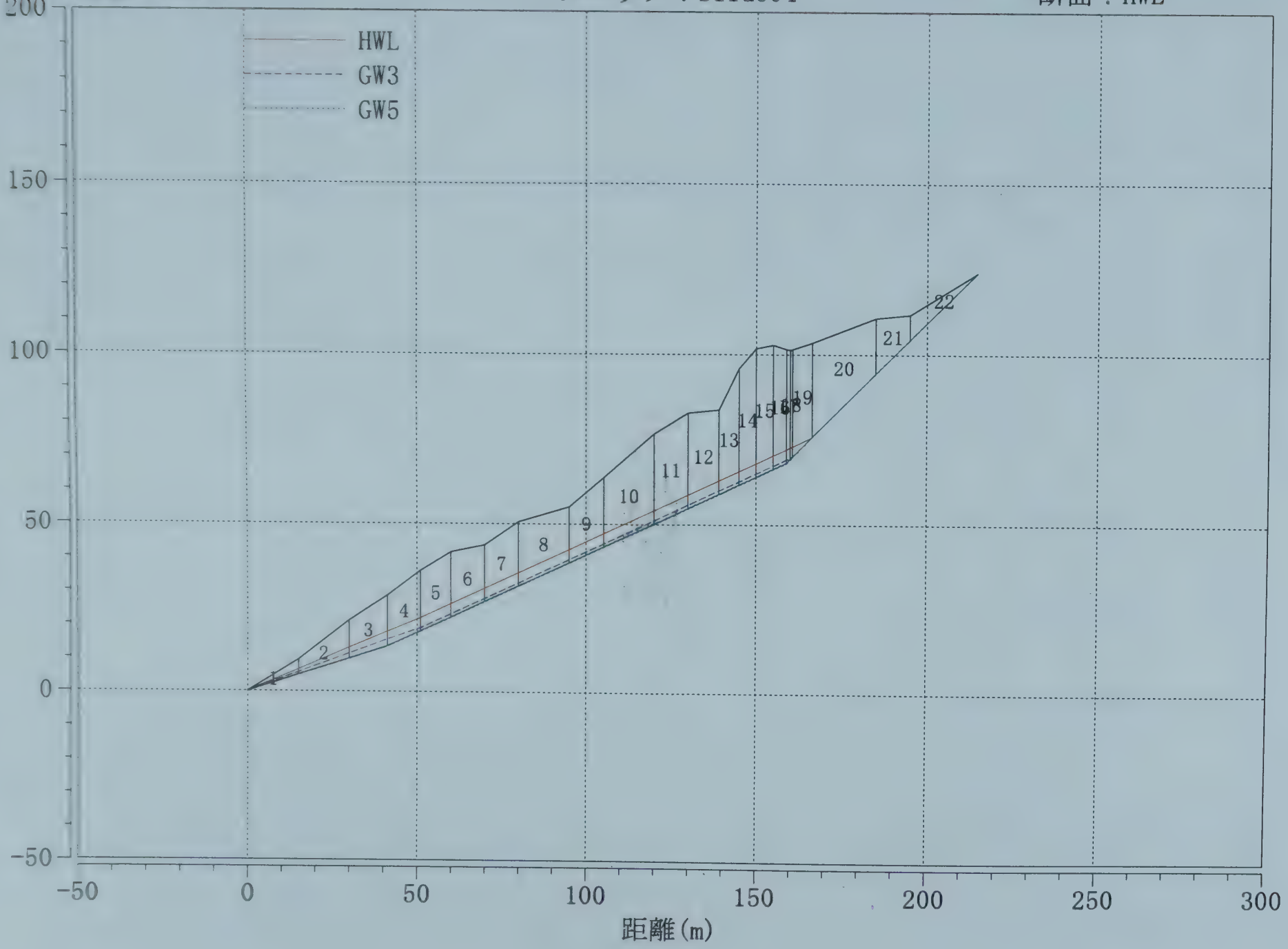
No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S = s_1 / n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1						0.0	3.93			82.95	1.0664	77.79	45.96
2						0.0	10.34			643.99	1.0664	603.92	356.79
3						0.0	3.17			237.30	1.0664	222.53	131.47
4						0.0	1.41			105.74	1.0664	99.16	58.58
5						0.0	1.52			142.17	1.0664	133.33	78.77
6						0.0	10.89			1611.12	1.0664	1510.86	892.60
7						0.0	11.03			3045.49	1.0664	2855.98	1687.28
8						0.0	5.94			2264.89	1.0664	2123.96	1254.81
9						0.0	4.80			1896.04	1.0664	1778.06	1050.46
10						0.0	3.94			1654.20	1.0664	1551.26	916.47
11						0.0	6.97			3234.57	1.0664	3033.30	1792.04
12						0.0	1.46			606.61	0.9469	640.66	658.35
13						0.0	4.86			1983.91	0.9469	2095.24	2153.11
14						0.0	17.22			6452.95	0.9469	6815.08	7003.32
15						0.0	14.66			4847.44	0.9469	5119.48	5260.88
16						0.0	9.47			2855.99	0.9469	3016.27	3099.58
17						0.0	8.19			2295.89	0.9469	2424.73	2491.70
18						0.0	15.93			4954.43	0.9469	5232.46	5376.98
19						0.0	20.00			7039.51	0.9469	7434.56	7639.91
20						0.0	4.98			1930.70	0.9469	2039.04	2095.36
21						0.0	2.55			1071.45	0.9469	1131.58	1162.84
22						0.0	2.40			988.81	0.9469	1044.30	1073.15
23						0.0	5.02			1942.70	0.9469	2051.73	2108.39
24						0.0	4.96			1713.14	0.9469	1809.28	1859.25
25						0.0	10.00			2827.86	0.9469	2986.56	3069.05
26						0.0	22.70			3395.42	0.9469	3585.97	3685.01
27						0.0	4.40			141.55	0.9469	149.50	153.63
28						0.0	1.68			10.11	0.9469	10.67	10.97

備考

Line-4

縦断面図

現場：NH717　　ブロック：Slide04　　断面：HWL　　縮尺：1/2000



A04-85

c’－φ’図

現場：NH717　　ブロック名：Slide04　　計算条件：

F_0 = 1.0000
c’ = 0.0000(kN/m²)
φ’ = 29.2419(°)

粘着力 c’(kN/m²)

250
200
150
100
50
0

0　5　10　15　20　25　30　35

内部摩擦角 φ’(°)

A04-86

Slide4_drainage & removal work

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide04	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター	平均値			末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m³)	26.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m³)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重量	γ_{sat} (kN/m³)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{ma} (kN/m)	
すべり面材料の粘着力	c' (kN/m²)	0.0000				杭 P_{mp} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	29.2419				その他 P_{mx} (kN/m)	
弦～すべり面の最大深さ	d (m)	20.41		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	248.05		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0430		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m²)	3819.81		二次元安全率		F_2	1.0000
地すべり抵抗力	Σ S (kN/m)	54354.45					
地すべり力	Σ T (kN/m)	56691.11					

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

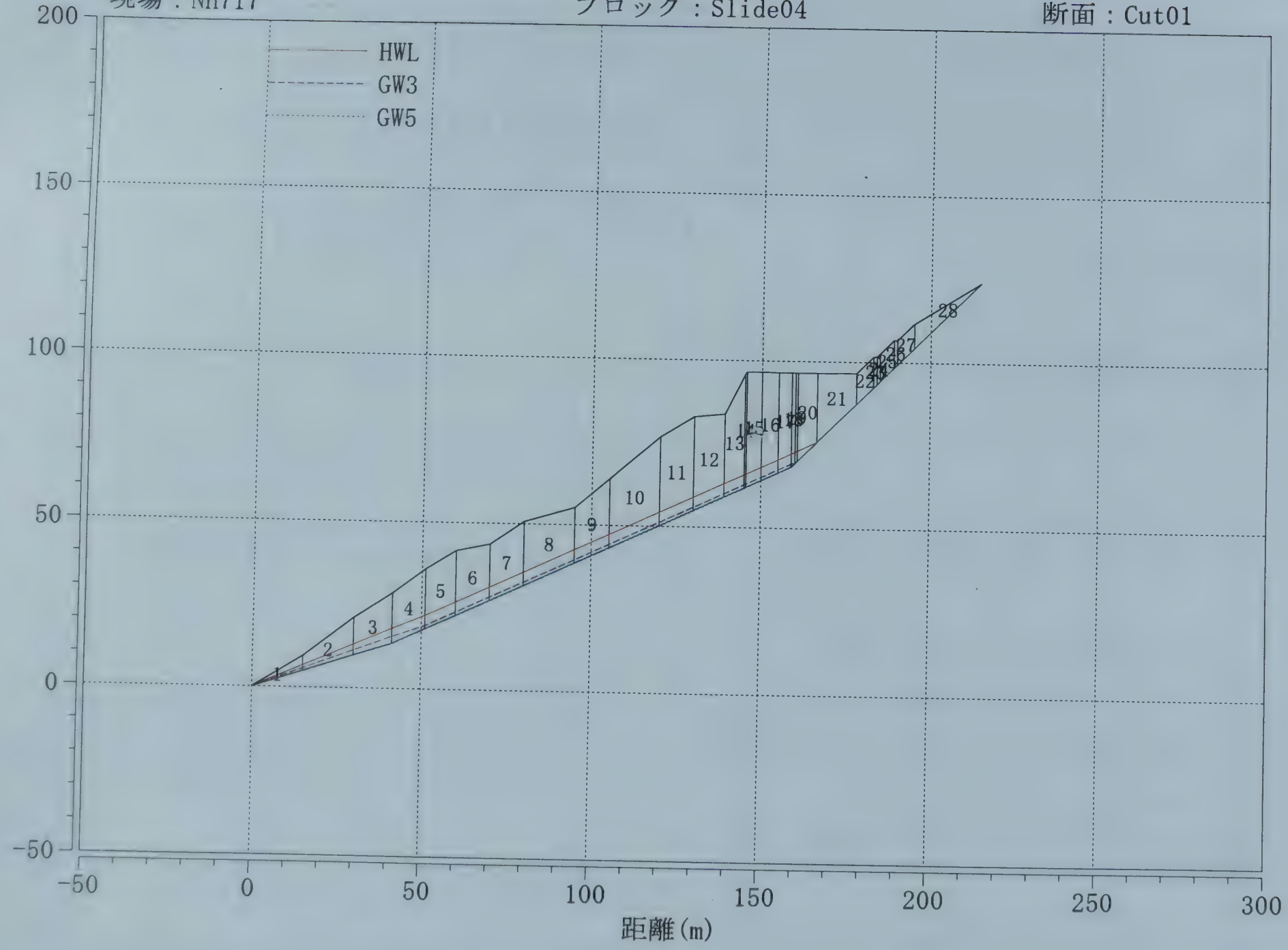
No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	4.520	15.00	18.000	881.40	15	37.670	36.310	5.00	25.000	4808.70
2	4.520	11.330	15.00	18.000	3090.75	16	36.310	33.270	3.92	25.000	3545.80
3	11.330	15.170	11.35	18.000	3910.08	17	33.270	31.960	1.08	45.000	915.83
4	15.170	18.000	9.72	25.000	4191.36	18	31.960	31.550	0.65	45.000	536.66
5	18.000	19.460	8.94	25.000	4353.60	19	31.550	27.700	5.78	45.000	4452.05
6	19.460	16.900	10.00	25.000	4726.80	20	27.700	16.390	18.57	45.000	10643.77
7	16.900	19.190	10.00	25.000	4691.70	21	16.390	7.390	10.00	45.000	3091.40
8	19.190	16.690	15.00	25.000	6996.60	22	7.390	0.000	19.75	45.000	1897.38
9	16.690	20.630	10.26	25.000	4977.74						
10	20.630	26.550	14.75	25.000	9046.77						
11	26.550	28.130	10.00	25.000	7108.40						
12	28.130	24.810	9.11	25.000	6269.68						
13	24.810	34.120	5.89	25.000	4512.27						
14	34.120	37.670	5.00	25.000	4666.35						

No	間隙水圧の高さ u_z (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	1.590	15.00				116.9	15.77			427.97	1.0690	400.33	286.38
2	3.170	15.00				350.1	15.77			1534.33	1.0690	1435.24	1004.25
3	4.370	11.35				419.6	11.93			1954.10	1.0690	1827.90	1270.46
4	3.900	9.72				394.2	10.72			2125.83	1.0358	2052.31	1954.46
5	3.900	8.94				341.9	9.86			2245.91	1.0358	2168.23	2030.12
6	3.900	10.00				382.5	11.03			2432.14	1.0358	2348.02	2204.14
7	3.900	10.00				382.5	11.03			2412.49	1.0358	2329.05	2187.78
8	3.900	15.00				573.7	16.55			3595.81	1.0358	3471.44	3262.57
9	3.900	10.26				392.4	11.32			2567.06	1.0358	2478.28	2321.16
10	3.900	14.75				564.1	16.27			4748.93	1.0358	4584.68	4218.58
11	3.900	10.00				382.5	11.03			3765.46	1.0358	3635.23	3314.70
12	3.900	9.11				348.4	10.05			3314.97	1.0358	3200.31	2923.60
13	3.900	5.89				225.3	6.50			2400.04	1.0358	2317.03	2104.11
14	3.900	5.00				191.2	5.52			2505.36	1.0358	2418.71	2175.95
15	3.900	5.00				191.2	5.52			2585.05	1.0358	2495.64	2242.33
16	3.900	3.92				149.9	4.33			1901.15	1.0358	1835.40	1653.43
17	3.340	1.08				38.3	1.53			491.25	0.7799	629.88	915.83
18	3.000	0.65				20.2	0.92			289.13	0.7799	370.72	536.66
19	0.000	5.78				85.0	8.17			2444.84	0.7799	3134.73	4452.05
20						0.0	26.26			5958.82	0.7799	7640.29	10643.77
21						0.0	14.14			1730.69	0.7799	2219.06	3091.40
22						0.0	27.93			1062.23	0.7799	1361.98	1897.38

備考：

縦断面図

現場：NH717　　ブロック：Slide04　　断面：Cut01　　縮尺：1/2000



A04-88

Slide4_drainage & removal work

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide04	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	Cut01	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター	平均値			末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m³)	26.000	
移動層の水中単位体積重量	γ' (kN/m³)		
移動層の飽和単位体積重量	γ_{sat} (kN/m³)		
陥没帯の内部摩擦角	ϕ_e (°)		
すべり面材料の粘着力	c' (kN/m²)	0.0000	
すべり面材料の内部摩擦角	ϕ' (°)	29.2419	
弦～すべり面の最大深さ	d (m)	20.41	
地すべり斜面弦長	L (m)	248.06	
修正係数	f_0	1.0430	
冠頭部クラックの傾斜角	θ_u (°)		
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		
移動層の断面積	A (m²)	3503.60	
地すべり抵抗力	ΣS (kN/m)	53112.98	
地すべり力	ΣT (kN/m)	49320.36	

設計済み負担推力				
分子側	アンカー	P_{sa} (kN/m)		
	杭	P_{sp} (kN/m)		
	その他	P_{sx} (kN/m)		
分母側	アンカー	P_{ma} (kN/m)		
	杭	P_{mp} (kN/m)		
	その他	P_{mx} (kN/m)		

クラックの水位高	Z_w (m)	
水圧の水平成分	Q_w (kN/m)	
陥没帯下流壁のすべり面深さ	Z_h (m)	
主働土圧係数	k_a	
主働土圧	E_a (kN/m)	
二次元安全率	F_2	1.1232

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	4.520	15.00	18.000	881.40	15	34.650	32.520	4.48	25.000	3911.98
2	4.520	11.330	15.00	18.000	3090.75	16	32.520	30.140	5.00	25.000	4072.90
3	11.330	15.170	11.35	18.000	3910.08	17	30.140	28.280	3.92	25.000	2977.08
4	15.170	18.000	9.72	25.000	4191.36	18	28.280	27.210	1.08	45.000	779.08
5	18.000	19.460	8.94	25.000	4353.60	19	27.210	26.560	0.65	45.000	454.36
6	19.460	16.900	10.00	25.000	4726.80	20	26.560	20.810	5.78	45.000	3559.38
7	16.900	19.190	10.00	25.000	4691.70	21	20.810	9.300	11.58	45.000	4532.76
8	19.190	16.690	15.00	25.000	6996.60	22	9.300	9.320	5.00	45.000	1210.30
9	16.690	20.630	10.26	25.000	4977.74	23	9.320	8.330	1.00	45.000	229.45
10	20.630	26.550	14.75	25.000	9046.77	24	8.330	8.330	1.00	45.000	216.58
11	26.550	28.130	10.00	25.000	7108.40	25	8.330	8.350	4.00	45.000	867.36
12	28.130	24.810	9.11	25.000	6269.68	26	8.350	7.360	1.00	45.000	204.23
13	24.810	34.120	5.89	25.000	4512.27	27	7.360	7.390	5.00	45.000	958.75
14	34.120	34.650	0.52	25.000	464.89	28	7.390	0.000	19.75	45.000	1897.38

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 ⊿x (kN/m)	地すべり抵抗力 s_1 (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 $S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1						0.0	15.77			493.44	1.0510	469.50	286.38
2						0.0	15.77			1730.33	1.0510	1646.37	1004.25
3						0.0	11.93			2189.02	1.0510	2082.81	1270.46
4						0.0	10.72			2346.50	1.0123	2317.97	1954.46
5						0.0	9.86			2437.33	1.0123	2407.70	2030.12
6						0.0	11.03			2646.26	1.0123	2614.09	2204.14
7						0.0	11.03			2626.61	1.0123	2594.68	2187.78
8						0.0	16.55			3916.99	1.0123	3869.37	3262.57
9						0.0	11.32			2786.74	1.0123	2752.87	2321.16
10						0.0	16.27			5064.75	1.0123	5003.18	4218.58
11						0.0	11.03			3979.58	1.0123	3931.20	3314.70
12						0.0	10.05			3510.03	1.0123	3467.36	2923.60
13						0.0	6.50			2526.15	1.0123	2495.45	2104.11
14						0.0	0.57			260.26	1.0123	257.10	216.78
15						0.0	4.94			2190.09	1.0123	2163.46	1824.19
16						0.0	5.52			2280.18	1.0123	2252.46	1899.22
17						0.0	4.33			1666.69	1.0123	1646.43	1388.24
18						0.0	1.53			436.16	0.7492	582.15	779.08
19						0.0	0.92			254.37	0.7492	339.51	454.36
20						0.0	8.17			1992.69	0.7492	2659.69	3559.38
21						0.0	16.38			2537.63	0.7492	3387.03	4532.76
22						0.0	7.07			677.58	0.7492	904.38	1210.30
23						0.0	1.41			128.46	0.7492	171.45	229.45
24						0.0	1.41			121.25	0.7492	161.84	216.58
25						0.0	5.66			485.58	0.7492	648.12	867.36
26						0.0	1.41			114.34	0.7492	152.61	204.23
27						0.0	7.07			536.75	0.7492	716.41	958.75
28						0.0	27.93			1062.23	0.7492	1417.79	1897.38

備考：

Line-5

縦断面図

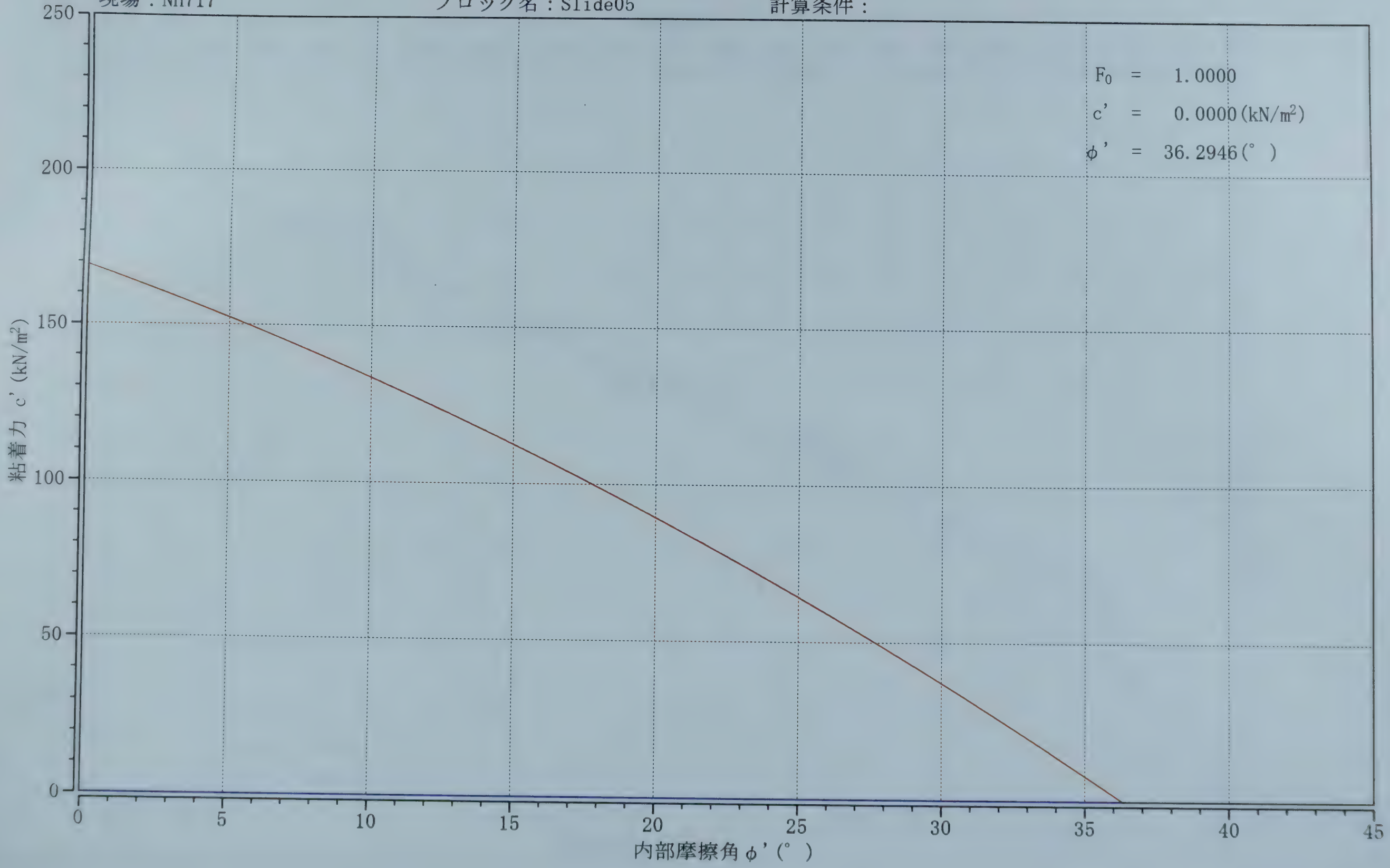
現場：NH717　　ブロック：Slide05　　断面：HWL　　縮尺：1/2000

A04-16-V01

c’－φ’図

現場：NH717　　ブロック名：Slide05　　計算条件：

F_0 = 1.0000
c’ = 0.0000(kN/m²)
φ’ = 36.2946(°)



A04-92

Slide5_drainage work.WS1

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide05	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		

移動層の湿潤単位体積重量	γ (kN/m^3)	26.000	
移動層の水中単位体積重量	γ' (kN/m^3)		
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)		
陥没帯の内部摩擦角	ϕ_e (°)		
すべり面材料の粘着力	c' (kN/m^2)	0.0000	
すべり面材料の内部摩擦角	ϕ' (°)	36.2946	
弦～すべり面の最大深さ	d (m)	40.48	
地すべり斜面弦長	L (m)	450.01	
修正係数	f_0	1.0458	
冠頭部クラックの傾斜角	θ_u (°)		
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)		
移動層の断面積	A (m^2)	7245.24	
地すべり抵抗力	Σ S (kN/m)	103575.65	
地すべり力	Σ T (kN/m)	108315.78	

設計済み負担推力	分子側	アンカー	P_{sa}	(kN/m)	
		杭	P_{sp}	(kN/m)	
		その他	P_{sx}	(kN/m)	
	分母側	アンカー	P_{ma}	(kN/m)	
		杭	P_{mp}	(kN/m)	
		その他	P_{mx}	(kN/m)	
クラックの水位高			Z_w	(m)	
水圧の水平成分			Q_w	(kN/m)	
陥没帯下流壁のすべり面深さ			Z_h	(m)	
主働土圧係数			k_a		
主働土圧			E_a	(kN/m)	
二次元安全率			F_2		1.0000

$$F_2 = f_0 \cdot \frac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$$

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	9.600	12.65	25.000	1578.72	36	26.630	33.180	7.17	25.000	5574.89
2	9.600	11.600	2.71	25.000	746.88	37	33.180	25.610	5.12	67.000	3913.06
3	11.600	14.600	5.53	25.000	1883.52	38	25.610	23.830	1.07	67.000	687.71
4	14.600	18.270	12.37	25.000	5285.82	39	23.830	21.400	1.60	67.000	940.78
5	18.270	17.480	11.89	25.000	5525.88	40	21.400	0.000	19.03	67.000	5294.15
6	17.480	19.590	15.91	25.000	7667.19						
7	19.590	24.690	18.93	25.000	10896.87						
8	24.690	25.330	15.32	25.000	9961.98						
9	25.330	27.110	5.02	25.000	3422.23						
10	27.110	26.400	5.02	25.000	3492.06						
11	26.400	21.900	20.07	25.000	12601.95						
12	21.900	20.970	10.03	25.000	5589.82						
13	20.970	22.500	11.44	25.000	6464.86						
14	22.500	22.870	3.61	25.000	2129.21						
15	22.870	20.740	9.49	25.000	5380.17						
16	20.740	19.490	21.54	25.000	11265.20						
17	19.490	17.170	9.11	25.000	4341.64						
18	17.170	14.600	17.57	25.000	7256.59						
19	14.600	11.970	12.53	25.000	4327.99						
20	11.970	13.170	25.09	25.000	8199.91						
21	13.170	11.600	3.89	25.000	1252.62						
22	11.600	11.130	1.13	25.000	333.90						
23	11.130	11.600	0.55	25.000	162.52						
24	11.600	14.600	3.37	25.000	1147.82						
25	14.600	15.440	0.91	25.000	355.37						
26	15.440	15.090	4.63	25.000	1837.60						
27	15.090	15.850	5.59	25.000	2248.41						
28	15.850	16.020	14.09	25.000	5837.63						
29	16.020	19.710	20.00	25.000	9289.80						
30	19.710	20.280	6.65	25.000	3457.14						
31	20.280	19.350	7.34	25.000	3781.49						
32	19.350	21.570	6.01	25.000	3197.08						
33	21.570	22.450	11.13	25.000	6369.25						
34	22.450	29.590	15.05	25.000	10181.63						
35	29.590	26.630	6.15	25.000	4494.79						

備考：

Slide5_drainage work.WS1

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide05	安定解析式	簡易Janbu法
断面名	HWL	間隙水圧名	HWL	地下水条件	被圧地下水
計算条件				水平外力 Q(kN/m)	0.00
				湛水条件	なし
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=
移動層の湿潤単位体積重量	γ (kN/m^3)	26.000			
移動層の水中単位体積重量	γ' (kN/m^3)				
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)				
陥没帯の内部摩擦角	ϕ_e (°)				
すべり面材料の粘着力	c' (kN/m^2)	0.0000			
すべり面材料の内部摩擦角	ϕ' (°)	36.2946			
地すべり抵抗力	Σ S (kN/m)	103575.65			
地すべり力	Σ T (kN/m)	108315.78			

陥没帯下流壁 No			
水平震度 k_h			
基準水面の標高BL(m)			
設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)
		杭	P_{sp} (kN/m)
		その他	P_{sx} (kN/m)
	分母側	アンカー	P_{ma} (kN/m)
		杭	P_{mp} (kN/m)
		その他	P_{mx} (kN/m)
移動層の断面積	A (m^2)	7245.24	
二次元安全率	F_2	1.0000	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_1 (kN/m)	n_α	$S=s_1/n_\alpha$ (kN/m)	地すべり力 T (kN/m)
1	9.600	12.65				595.5	13.96			722.13	1.1027	654.88	736.17
2	11.600	2.71				281.7	2.99			341.63	1.1027	309.82	348.27
3	14.600	5.53				710.4	6.10			861.55	1.1027	781.32	878.30
4	14.600	12.37				1771.1	13.65			2581.31	1.1027	2340.91	2464.82
5	14.600	11.89				1702.4	13.12			2808.09	1.1027	2546.56	2576.76
6	14.600	15.91				2277.9	17.55			3958.01	1.1027	3589.39	3575.27
7	14.600	18.93				2710.3	20.89			6012.41	1.1027	5452.46	5081.29
8	14.600	15.32				2193.5	16.90			5705.41	1.1027	5174.06	4645.35
9	14.600	5.02				718.7	5.54			1985.52	1.1027	1800.60	1595.81
10	14.600	5.02				718.7	5.54			2036.80	1.1027	1847.11	1628.38
11	14.600	20.07				2873.6	22.14			7144.80	1.1027	6479.39	5876.39
12	14.600	10.03				1436.1	11.07			3050.63	1.1027	2766.52	2606.58
13	14.600	11.44				1637.9	12.62			3545.02	1.1027	3214.87	3014.61
14	14.600	3.61				516.9	3.98			1184.15	1.1027	1073.87	992.87
15	14.600	9.49				1358.8	10.47			2953.44	1.1027	2678.38	2508.81
16	14.600	21.54				3084.0	23.77			6008.48	1.1027	5448.90	5253.05
17	14.600	9.11				1304.3	10.05			2230.68	1.1027	2022.93	2024.54
18	14.600	17.57				2515.6	19.39			3481.90	1.1027	3157.62	3383.80
19	11.970	12.53				1632.4	13.83			1979.70	1.1027	1795.32	2018.17
20	13.170	25.09				3092.8	27.68			3750.78	1.1027	3401.47	3823.68
21	11.600	3.89				472.5	4.29			572.97	1.1027	519.61	584.11
22	11.130	1.13				125.9	1.25			152.73	1.1027	138.51	155.70
23	11.600	0.55				61.3	0.61			74.34	1.1027	67.42	75.78
24	14.600	3.37				432.9	3.72			525.03	1.1027	476.14	535.24
25	14.600	0.91				130.3	1.00			165.31	1.1027	149.91	165.71
26	14.600	4.63				662.9	5.11			862.73	1.1027	782.38	856.89
27	14.600	5.59				800.4	6.17			1063.49	1.1027	964.44	1048.45
28	14.600	14.09				2017.4	15.55			2805.71	1.1027	2544.41	2722.13
29	14.600	20.00				2863.5	22.07			4719.62	1.1027	4280.08	4331.90
30	14.600	6.65				952.1	7.34			1839.75	1.1027	1668.41	1612.09
31	14.600	7.34				1050.9	8.10			2005.41	1.1027	1818.64	1763.34
32	14.600	6.01				860.5	6.63			1716.05	1.1027	1556.23	1490.82
33	14.600	11.13				1593.6	12.28			3507.40	1.1027	3180.75	2970.03
34	14.600	15.05				2154.8	16.61			5895.11	1.1027	5346.09	4747.77
35	14.600	6.15				880.5	6.79			2654.41	1.1027	2407.20	2095.95
36	14.600	7.17				1026.6	7.91			3340.41	1.1027	3029.31	2599.61
37	5.000	5.12				492.1	13.10			2512.48	0.4168	6027.70	9218.60
38	3.000	1.07				42.0	2.74			474.25	0.4168	1137.77	1620.14
39	0.000	1.60				23.5	4.09			673.65	0.4168	1616.16	2216.35
40						0.0	48.70			3888.17	0.4168	9328.12	12472.23

備考：

Slide5_drainage work.WS1

二　次　元　安　定　解　析　表

現場名	NH717	ブロック名	Slide05	安定解析式	簡易Janbu法	陥没帯下流壁 No.	
断面名	HWL	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	26.000		設計済み負担推力	分子側	アンカー	P_{sa} (kN/m)
移動層の水中単位体積重量	γ' (kN/m^3)					杭	P_{sp} (kN/m)
移動層の飽和単位体積重量	γ_{sat} (kN/m^3)					その他	P_{sx} (kN/m)
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー	P_{ma} (kN/m)
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭	P_{mp} (kN/m)
すべり面材料の内部摩擦角	ϕ' (°)	36.2946				その他	P_{mx} (kN/m)
弦～すべり面の最大深さ	d (m)	40.48		クラックの水位高		Z_w (m)	
地すべり斜面弦長	L (m)	450.01		水圧の水平成分		Q_w (kN/m)	
修正係数	f_0	1.0458		陥没帯下流壁のすべり面深さ		Z_h (m)	
冠頭部クラックの傾斜角	θ_u (°)			主働土圧係数		k_a	
陥没帯下流壁ｸﾗｯｸの傾斜角	θ_d (°)			主働土圧		E_a (kN/m)	
移動層の断面積	A (m^2)	7245.24		二次元安全率		F_2	1.1589
地すべり抵抗力	Σ S (kN/m)	120028.35		$F_2 = f_0 \cdot \dfrac{\Sigma S + P_{sa} + P_{sp} + P_{sx}}{\Sigma T + Q_w + E_a - P_{ma} - P_{mp} - P_{mx} + Q}$			
地すべり力	Σ T (kN/m)	108315.78					

No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)	No	下流端のすべり面深さ h_1 (m)	上流端のすべり面深さ h_2 (m)	スライス幅 b (m)	すべり面傾斜角 α (°)	重量 W (kN/m)
1	0.000	9.600	12.65	25.000	1578.72	36	26.630	33.180	7.17	25.000	5574.89
2	9.600	11.600	2.71	25.000	746.88	37	33.180	25.610	5.12	67.000	3913.06
3	11.600	14.600	5.53	25.000	1883.52	38	25.610	23.830	1.07	67.000	687.71
4	14.600	18.270	12.37	25.000	5285.82	39	23.830	21.400	1.60	67.000	940.78
5	18.270	17.480	11.89	25.000	5525.88	40	21.400	0.000	19.03	67.000	5294.15
6	17.480	19.590	15.91	25.000	7667.19						
7	19.590	24.690	18.93	25.000	10896.87						
8	24.690	25.330	15.32	25.000	9961.98						
9	25.330	27.110	5.02	25.000	3422.23						
10	27.110	26.400	5.02	25.000	3492.06						
11	26.400	21.900	20.07	25.000	12601.95						
12	21.900	20.970	10.03	25.000	5589.82						
13	20.970	22.500	11.44	25.000	6464.86						
14	22.500	22.870	3.61	25.000	2129.21						
15	22.870	20.740	9.49	25.000	5380.17						
16	20.740	19.490	21.54	25.000	11265.20						
17	19.490	17.170	9.11	25.000	4341.64						
18	17.170	14.600	17.57	25.000	7256.59						
19	14.600	11.970	12.53	25.000	4327.99						
20	11.970	13.170	25.09	25.000	8199.91						
21	13.170	11.600	3.89	25.000	1252.62						
22	11.600	11.130	1.13	25.000	333.90						
23	11.130	11.600	0.55	25.000	162.52						
24	11.600	14.600	3.37	25.000	1147.82						
25	14.600	15.440	0.91	25.000	355.37						
26	15.440	15.090	4.63	25.000	1837.60						
27	15.090	15.850	5.59	25.000	2248.41						
28	15.850	16.020	14.09	25.000	5837.63						
29	16.020	19.710	20.00	25.000	9289.80						
30	19.710	20.280	6.65	25.000	3457.14						
31	20.280	19.350	7.34	25.000	3781.49						
32	19.350	21.570	6.01	25.000	3197.08						
33	21.570	22.450	11.13	25.000	6369.25						
34	22.450	29.590	15.05	25.000	10181.63						
35	29.590	26.630	6.15	25.000	4494.79						

備考：

Slide5_drainage work.WS1

二 次 元 安 定 解 析 表

現場名	NH717	ブロック名	Slide05	安定解析式	簡易Janbu法	陥没帯下流壁 No	
断面名	HWL	間隙水圧名	GW5	地下水条件	被圧地下水		
計算条件				水平外力 Q(kN/m)	0.00	水平震度 k_h	
				湛水条件	なし	基準水面の標高BL(m)	
土質パラメーター		平均値		末端座標 (m)	x= , y= , z=		
移動層の湿潤単位体積重量	γ (kN/m^3)	26.000		設計済み負担推力	分子側	アンカー P_{sa} (kN/m)	
移動層の水中単位体積重量	γ' (kN/m^3)					杭 P_{sp} (kN/m)	
移動層の飽和単位体積重力	γ_{sat} (kN/m^3)					その他 P_{sx} (kN/m)	
陥没帯の内部摩擦角	ϕ_e (°)				分母側	アンカー P_{aa} (kN/m)	
すべり面材料の粘着力	c' (kN/m^2)	0.0000				杭 P_{ap} (kN/m)	
すべり面材料の内部摩擦角	ϕ' (°)	36.2946				その他 P_{ax} (kN/m)	
地すべり抵抗力	Σ S (kN/m)	120028.35		移動層の断面積	A (m^2)	7245.24	
地すべり力	Σ T (kN/m)	108315.78		二次元安全率	F_2	1.1589	

No	間隙水圧の高さ u_2 (m)	間隙水圧対応の幅 b_u (m)	すべり面法線分力 N (kN/m)	浸透水圧の傾斜角 θ_p (°)	浸透水圧 P (kN/m)	間隙水圧 U (kN/m)	すべり面長 L (m)	推力線の傾き α_t (°)	鉛直方向せん断力 Δx (kN/m)	地すべり抵抗力 s_l (kN/m)	地すべり抵抗力 n_α	地すべり抵抗力 S=s_l / n_α (kN/m)	地すべり力 T (kN/m)
1	9.600	12.65				595.5	13.96			722.13	1.0641	678.61	736.17
2	9.600	2.71				255.1	2.99			361.15	1.0641	339.38	348.27
3	9.600	5.53				520.6	6.10			1000.95	1.0641	940.62	878.30
4	9.600	12.37				1164.6	13.65			3026.77	1.0641	2844.34	2464.82
5	9.600	11.89				1119.4	13.12			3236.26	1.0641	3041.21	2576.76
6	9.600	15.91				1497.8	17.55			4530.95	1.0641	4257.86	3575.27
7	9.600	18.93				1782.1	20.89			6694.11	1.0641	6290.64	5081.29
8	9.600	15.32				1442.3	16.90			6257.11	1.0641	5879.98	4645.35
9	9.600	5.02				472.6	5.54			2166.29	1.0641	2035.73	1595.81
10	9.600	5.02				472.6	5.54			2217.58	1.0641	2083.92	1628.38
11	9.600	20.07				1889.5	22.14			7867.55	1.0641	7393.36	5876.39
12	9.600	10.03				944.3	11.07			3411.83	1.0641	3206.19	2606.58
13	9.600	11.44				1077.0	12.62			3956.99	1.0641	3718.49	3014.61
14	9.600	3.61				339.9	3.98			1314.15	1.0641	1234.95	992.87
15	9.600	9.49				893.4	10.47			3295.19	1.0641	3096.58	2508.81
16	9.600	21.54				2027.9	23.77			6784.17	1.0641	6375.27	5253.05
17	9.600	9.11				857.7	10.05			2558.74	1.0641	2404.52	2024.54
18	9.600	17.57				1654.1	19.39			4114.62	1.0641	3866.62	3383.80
19	9.600	12.53				1179.6	13.83			2312.25	1.0641	2172.88	2018.17
20	9.600	25.09				2362.1	27.68			4287.48	1.0641	4029.06	3823.68
21	9.600	3.89				366.2	4.29			651.00	1.0641	611.76	584.11
22	9.600	1.13				106.4	1.25			167.10	1.0641	157.03	155.70
23	9.600	0.55				51.8	0.61			81.33	1.0641	76.43	75.78
24	9.600	3.37				317.3	3.72			609.98	1.0641	573.22	535.24
25	9.600	0.91				85.7	1.00			198.08	1.0641	186.14	165.71
26	9.600	4.63				435.9	5.11			1029.46	1.0641	967.41	856.89
27	9.600	5.59				526.3	6.17			1264.79	1.0641	1188.56	1048.45
28	9.600	14.09				1326.5	15.55			3313.11	1.0641	3113.42	2722.13
29	9.600	20.00				1882.9	22.07			5439.85	1.0641	5111.98	4331.90
30	9.600	6.65				626.1	7.34			2079.22	1.0641	1953.90	1612.09
31	9.600	7.34				691.0	8.10			2269.73	1.0641	2132.93	1763.34
32	9.600	6.01				565.8	6.63			1932.48	1.0641	1816.01	1490.82
33	9.600	11.13				1047.8	12.28			3908.21	1.0641	3672.65	2970.03
34	9.600	15.05				1416.9	16.61			6437.09	1.0641	6049.11	4747.77
35	9.600	6.15				579.0	6.79			2875.88	1.0641	2702.54	2095.95
36	9.600	7.17				675.0	7.91			3598.61	1.0641	3381.71	2599.61
37	0.000	5.12				241.0	13.10			2696.86	0.3806	7085.53	9218.60
38						0.0	2.74			505.07	0.3806	1326.99	1620.14
39						0.0	4.09			690.94	0.3806	1815.32	2216.35
40						0.0	48.70			3888.17	0.3806	10215.49	12472.23

備考：

Appendix V

Reference on Gravity Deformation & Textbook for Map Reading

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CHAPTER 1 Reference on Gravity Deformation

1.1 Cases of Gravity Deformation / Deep-seated Landslide

1.1.1 Gravity Deformation and Deep-seated Landslide in Japan

In the Akaishi Mountains, also known as the Southern Alps stretching across three prefectures in Japan namely Nagano, Yamanashi, and Shizuoka, and called as the Japanese Alps along with the Hida Mountains (Northern Alps) and the Kiso Mountains (Central Alps), a series of linear depressions stretch on the wide ridge on the southern side of Mt. Ainodake, one of the main peaks of the Southern Alps. Landslide engineers in Japan consider these linear depressions to have been formed by displacement or fractured zones caused by faulting of the fragile shale of Mt. Ainodake. In this section, a representative study by Matsuoka (1985) on linear depression are introduced. Linear depression is a characteristic microtopography of gravity deformation and also explained in Figure 1-23 of JICA Slope Protection Guideline.

Matsuoka (1985) reported distribution of linear depressions and lithological characteristic at main ridge of the Akaishi Range along with their relationship, and studied the effects of lithological characteristic on formation of fault topography. From his paper, descriptions regarding formation of gravity deformation are summarized below.

In ideally isotropic and homogeneous rock mass, strength and direction of stress given to rock mass govern directions of fractures developed in rock mass. In anisotropic rock mass where bedding planes and joints develop, however, directions of fractures are affected by anisotropies as well as strength and direction of stress.

In addition, even under isotropic stress, yield strengths depend on rock types.

Picture 1-1 Linear Depression Developing on Southern Ridgeline of Mt. Ainodake

Source: Geosites in Yamanashi、http://walstone.sub.jp/giy/giy02/13081205.html

Linear depressions are generally found along ridges of mountains in a manner parallel to nearly parallel to the ridges. If ridges bend, linear depressions often bend as well. These facts

indicate that formation of linear depressions is controlled by local stress fields around ridgelines rather than those of wide areas.

If distribution of linear depressions was simply controlled by local stress field around ridgeline, linear depressions should be evenly distributed and extending along any ridges of any directions. Actual distribution of linear depressions are, however, limited. Following relationships are indicated between distribution of linear depressions and properties of bedrock that forms ridgeline.

- At the site where linear depressions develop, directions of ridgeline are often parallel to the strike of bedding planes (Paschinger (1928)).
- Linear depressions are often formed parallel to the strikes of bedding planes or joints (Jan (1964)).
- Linear depressions appear more frequently in the areas where bedrock is composed of sedimentary rock or crystalline schist than in the areas of granite (Jahn (1964)).
- Linear depressions are frequently formed in the areas of sedimentary rock and crystalline schist. On the other hand, linear depressions are not frequently found in the areas of granite, hard and massive lower Paleozoic sedimentary rock, or Tertiary sedimentary soft rock (Beck, 1968).
- Linear depressions develop more frequently in the area of sedimentary rock than volcanic rock (Tabor, 1971).

According to the above-mentioned studies, linear depressions are often developed along the ridgeline where hard sedimentary rock with layer structure or crystalline schist lies and their bedding plane or schistosity is parallel to the ridgeline. Thus not only stress field but properties of bedrock around ridgeline affect development of linear depressions.

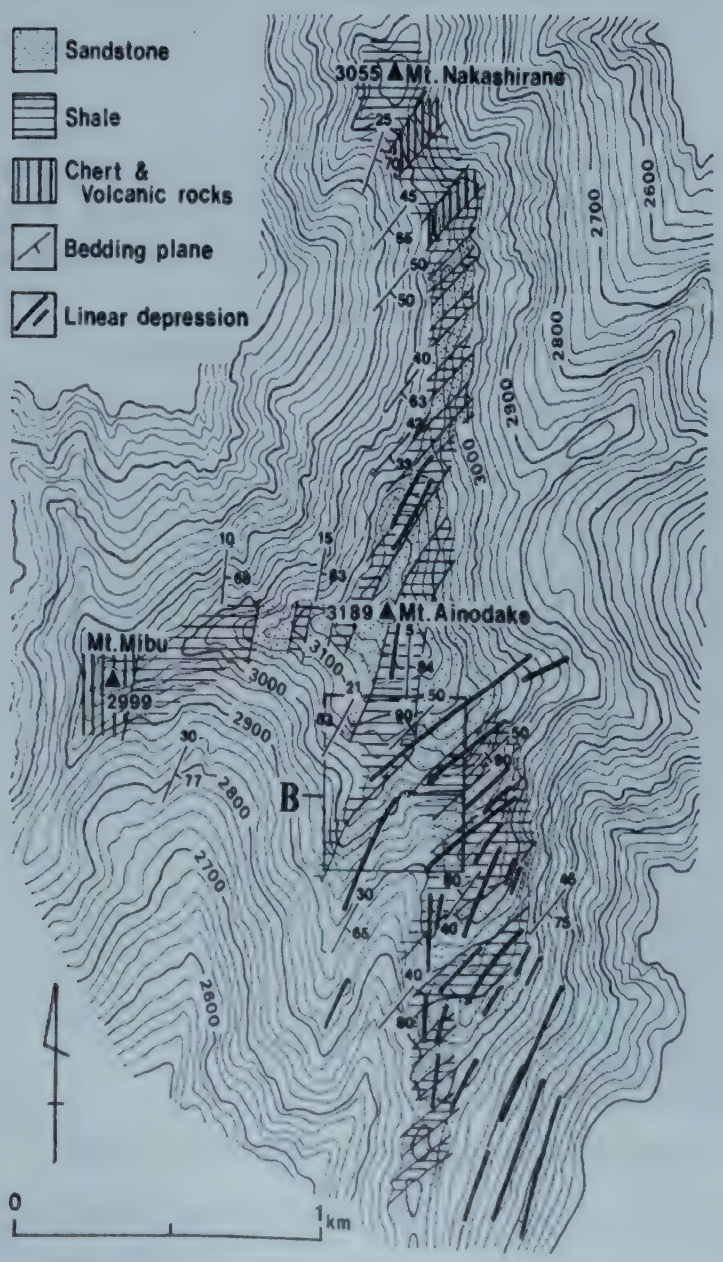


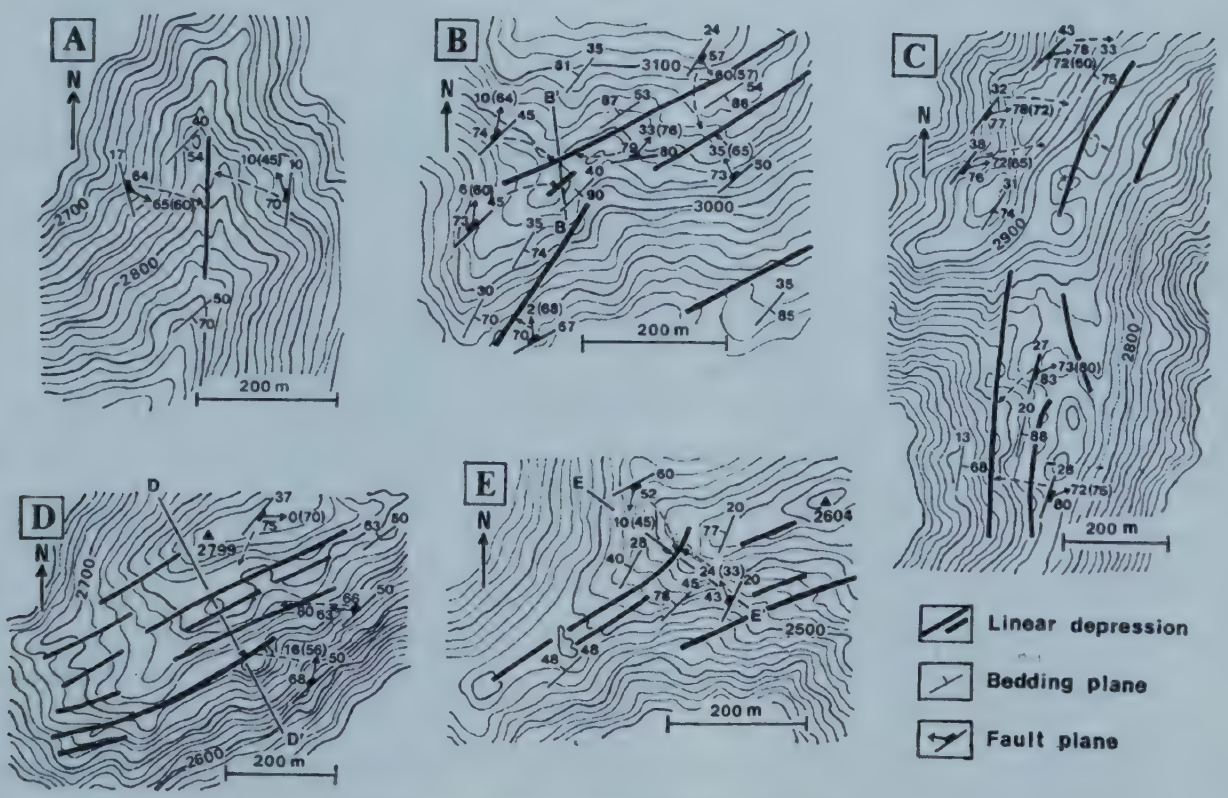
Figure 1-1 Distribution of Linear Depressions and Geological Conditions around Mt. Ainodake (contour interval: 20 m)

Source : Matsuoka (1985)

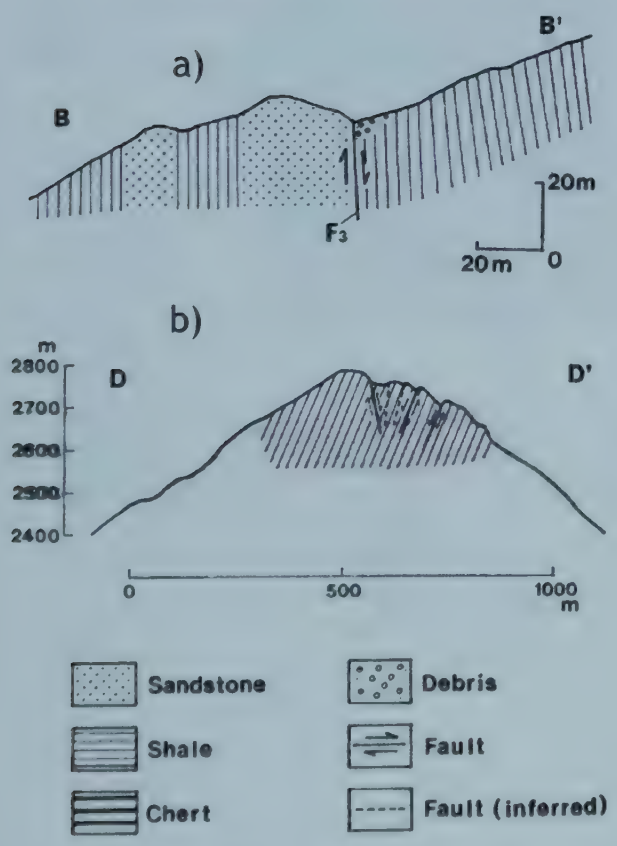
Figure 1-1 shows relationship between linear depressions and geological conditions along the ridgeline of Mt. Ainodake. On the southern ridge of Mt. Ainodake, the gentle slope is formed in the shape of several steps (Matsuoka & Ono, 1982). On the gentle slopes, a series of linear depressions are concentrated in a manner forming multiple ridges. The predominant directions of the linear depressions are N-S and NE-SW. The former almost corresponds to the main ridgeline, and the latter corresponds to the strike of the bedding planes. In addition, the linear depressions around Mt. Ainodake distinctively develops in the strata of shale than in sandstone.

Some undulations of the ridgelines of the Akaishi Range, which is under dissection, are well correspond to differences of rock types. Especially, shale appears on the cols on the ridge. Based on the aforementioned observations, Matsuoka and Uemoto (1984) initially inferred that differential erosion formed these undulations. As most of the

linear depressions are completely closed, however, deposited materials produced by erosion have no outlet to be drained. Matsuoka and Uemoto (1984) concluded that, therefore, faults formed the linear depressions in the Akaishi Range and the geological structure constrained development of such faults.



(i) Linear Depressions on Topo Map



(ii) Cross-section of Linear Depression

(iii) Linear Depression on the South Slope of Mt. Ainodake (Black Arrow Indicates Fault F3)

Figure 1-2 Relationship between Linear Depressions and Faults in the Akaishi Mountains

Source: Matsuoka (1985)

Figure 1-2 (i)-B explains reverse scarp type linear depressions formed on the southern slope of Mt. Ainodake. A reverse scarp type linear depression typically has scarplets facing toward ridgelines. The directions of the linear depressions are similar to those of the strikes of the strata. A fault, shown as F3 on cross-section B-B' in Figure 1-2 (ii)-a, appears on the inner surface of the linear depression as shown in Figure 1-2 (iii). The fault plane is almost parallel to the bedding of the strata, with the dip toward the ridgeline with a gradient of 70 to 90 degrees.

Linear depressions prone to develop where geological strata intersect slope surface with almost right angle. And in many cases, reverse scarp type linear depressions tend to appear unevenly on the either side of the ridgeline as shown in Figure 1-2 (ii)-b.

Above mentioned discussion lead the following consideration:

- Linear depressions are formed mainly by displacement of normal faults.
- Fault plane of normal fault often corresponds to bedding plane of geological strata.

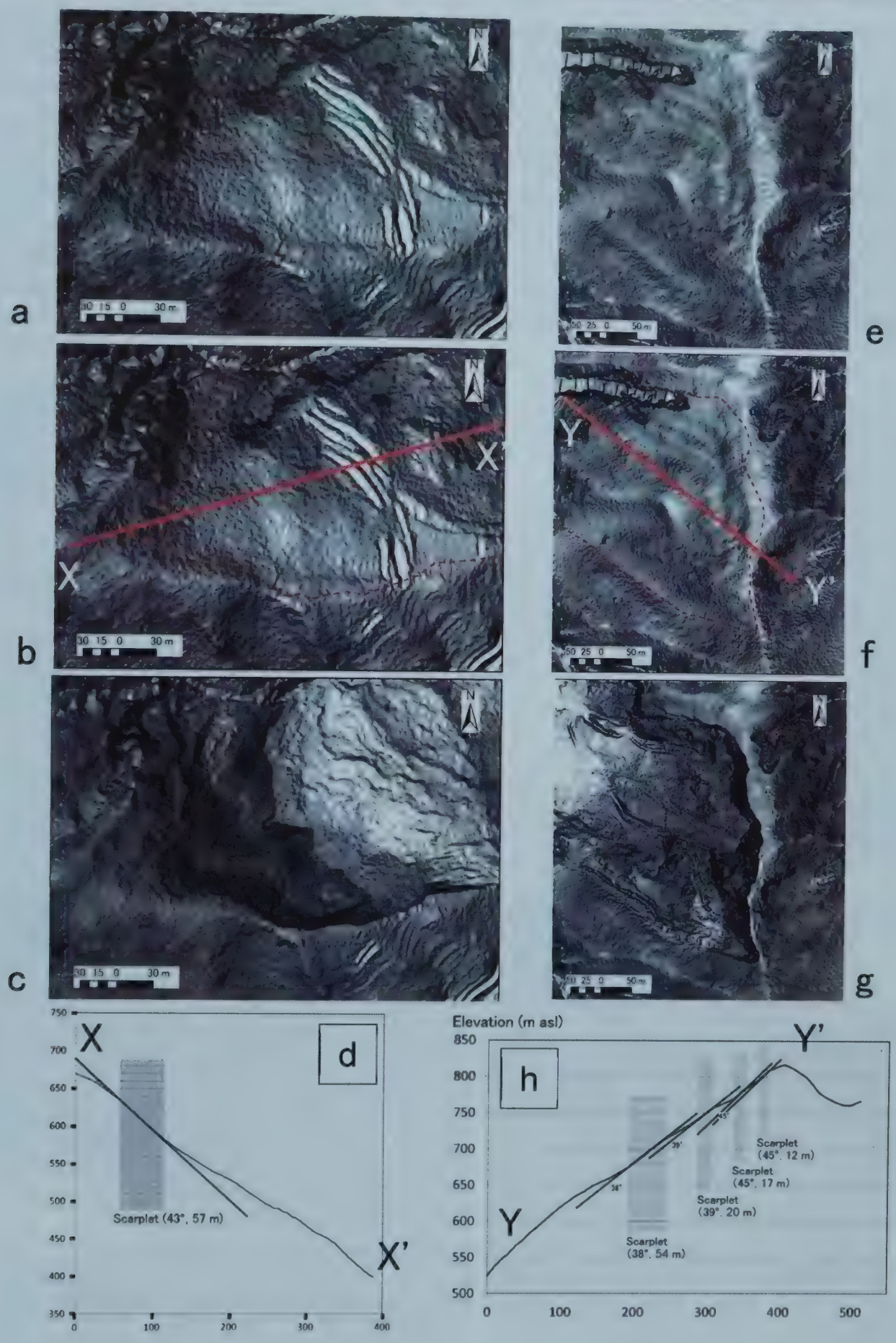
Local stress field around a ridgeline forms normal faults along a ridgeline, with tensile stress toward the steepest direction of slopes along a ridgeline, formed by gravity.

From the 2nd to 5th of September, 2011, Typhoon 1112 caused devastating damage in Kii Peninsula including deep-seated landslides, with its tremendous rainfall reaching more than 2,000 mm in accumulated amount. More than 50 deep-seated catastrophic landslides occurred in the Kii Mountains, with affected areas ranging between 36,000 and 549,000 m^2. The maximum volume of displaced mass is estimated to be 15 million m^3.

Amongst them, Chigira et al. (2012) studied fourteen landslides by using 1m-DEMs or aerial photographs, and reported that most of the studied landslides occurred on the slopes where scarplets or linear depressions had been observed beforehand; the sraplets or linear depressions correspond to the crowns of the deep-seated landslides as shown in Figure 1-3. These scarplets were 2 to 50 meters high. Horizontal length ratio between the scarplets and the deep-seated landslides on the profile along the steepest slope direction after the failure falls between 5 and 21 %. Gradient between the crown and the toe of the landslides ranges between 27 and 34 degrees.

The scarplets observed before the landslides in 2011 and mentioned by Chigira et al. correspond to critical strain before deep-seated landslides occur.

Without particular attention, however, these scarplets are scarcely observed through aerial photo readings. In order to extract them objectively and without errors, detailed DEMs (high density mesh with 1 meter or less) measured by aerial laser profiler are required.



Slope images before (a, b, e, and f) and after (c and g) the event of 2011 and Cross sections (d and h)
Ui landslide (a, b, c and d) and Nagatono landslide (e, f, g and h)

Figure 1-3 Scarplets Detected by Detailed DEMs of Ui and Nagatono Landslides

Source: Chigira, et al. (2012)

As explained in the aforementioned study by Chigira et al., in Japan, a block of gravity deformation, which is recognized by scarplets and linear depressions, is considered as a precursors of a deep-seated landslide. Scarplets and linear depressions are, therefore, considered as key topographic features when extracting slopes with high risks of deep-seated landslides.

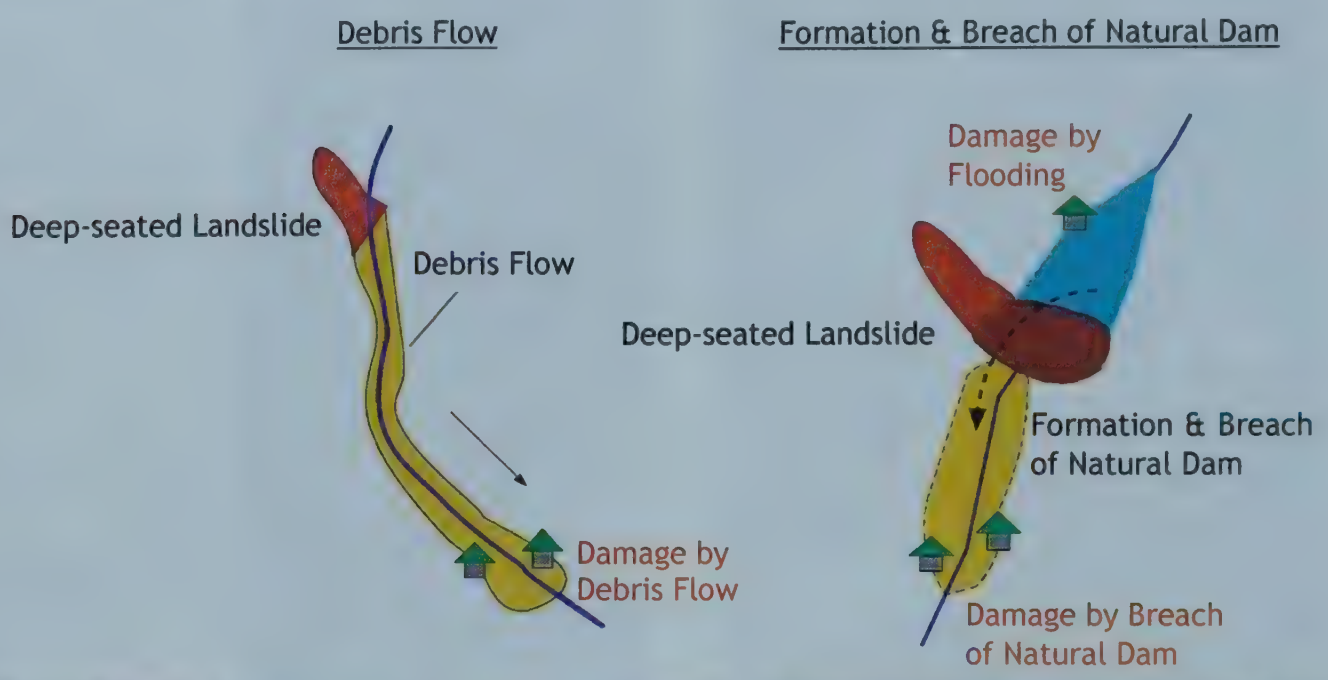
Although deep-seated landslides occur less frequently than debris flows or general slope failures do, the damage caused by a deep-seated landslide is much more catastrophic once it occurs.

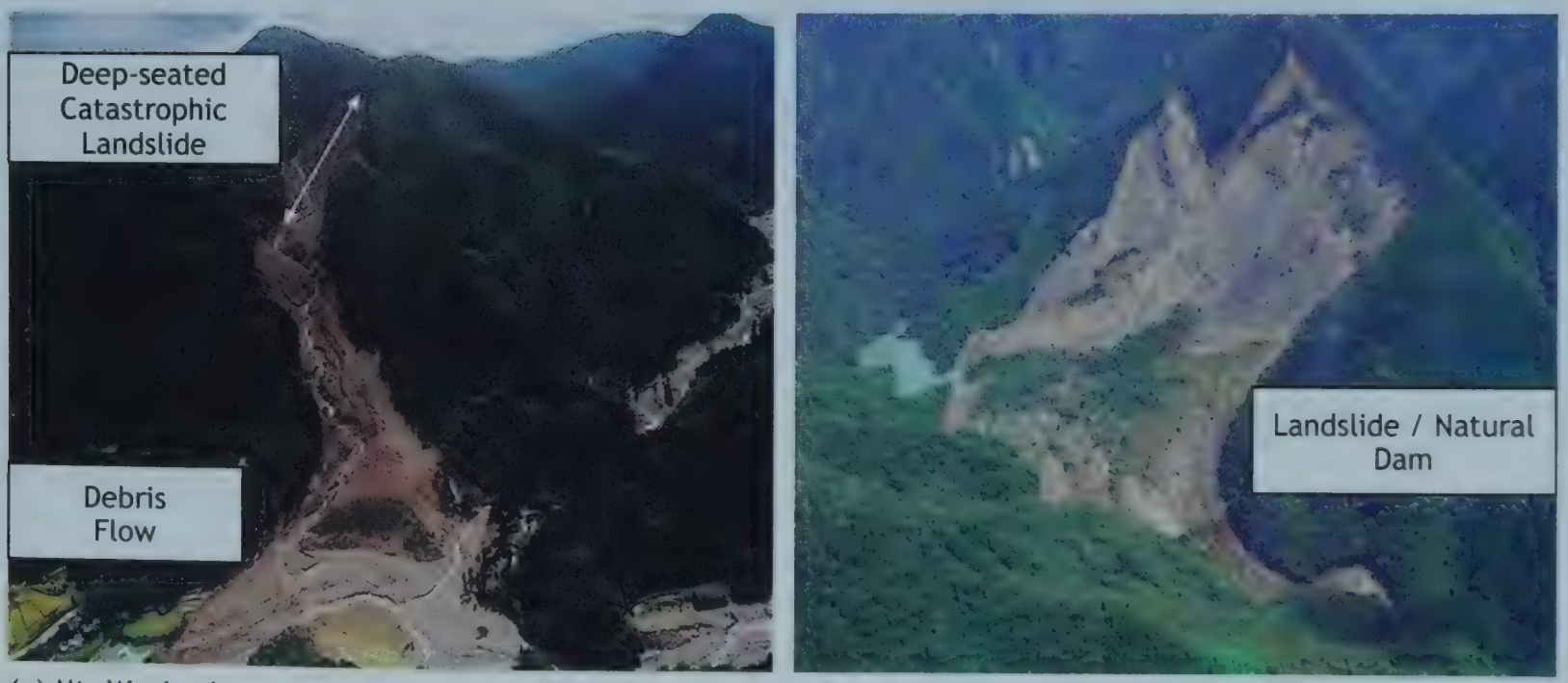
Two types of damage are known due to deep-seated landslide. One is direct hit by huge collapsed mass of deep-seated landslide, and another is debris flow generated by collapsed mass falling into a river running at the foot of the slope. Such debris flows often attack villages downstream and cause devastating damage.

Deep-seated landslide causes large scale debris flow in two ways.

By falling into a stream or a river, a collapsed mass obtains fluidity and turns into a debris flow, which flows downward along the stream or river.

In case a collapsed mass is huge, it closes a river channel and forms a natural dam or a landslide dam. In such case, flooding can be caused upstream and, if a natural dam collapses, debris flows cause serious damage downstream.





(a) Mt. Wanitsuka, Miyazaki pref. (occurred in 2005)

(b) Kuridaira, Totsukawa Village, Nara Pref. (occurred in 2005)

Left: debris flow Right: formation of landslide dam

Figure 1-4 Debris Flow and Landslide Dam Caused by Deep-seated Landslides

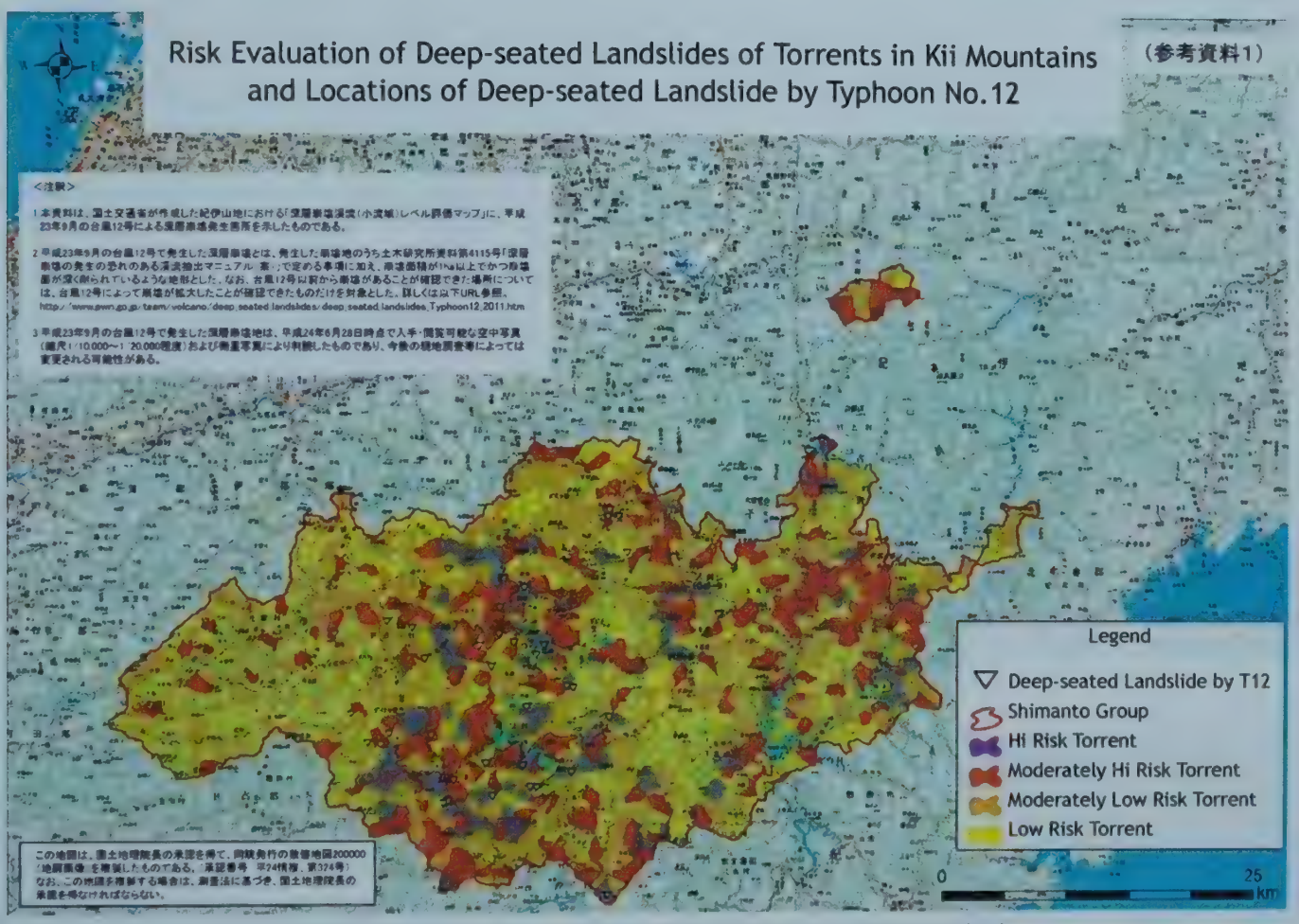
Source: Goto (2012)

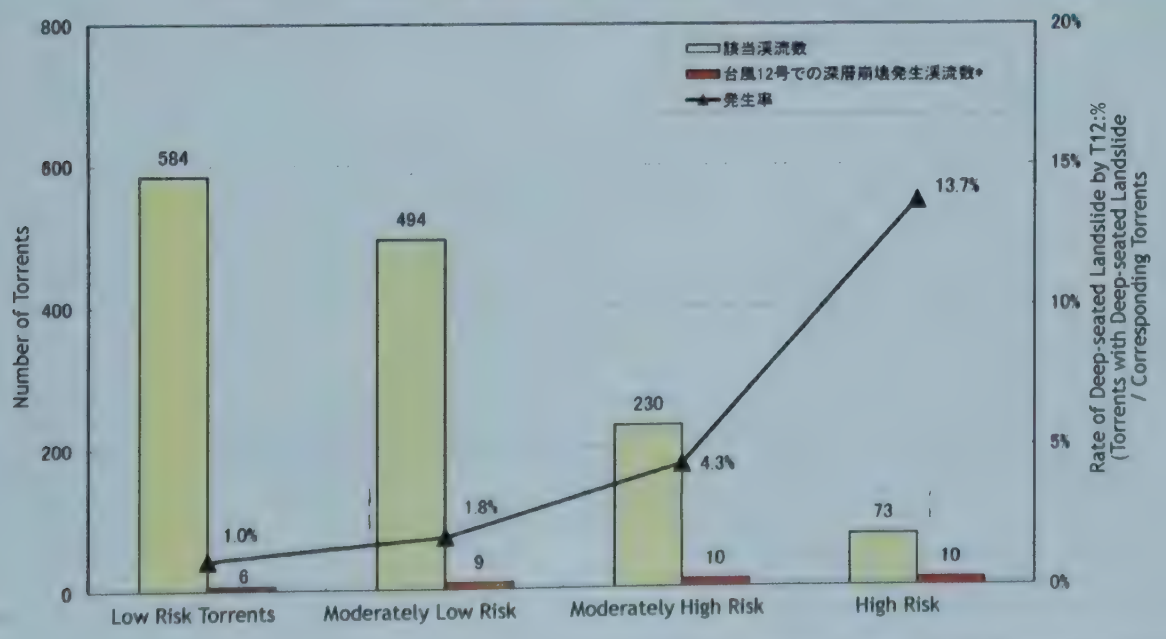
In Japan, the Ministry of Land, Infrastructure Transport and Tourism (MLIT) has took an initiative in carrying out a series of intensive investigation on deep-seated landslides. As a result, "Estimated frequency map for deep-seated landslides" was published in August 2010, which was followed by "Risk level map for deep-seated landslide prone mountain streams" and "Density map for traces of deep-seated landslide" published in September 2012. In addition, to reinforce early

warning and evacuation system, in the areas where deep-seated landslides are highly probable, acceleration sensors are deployed to detect an occurrence of a massive failure.

At present, however, factors and triggers, and estimation of degree and affected area to be damaged by a deep-seated landslide require further studies. In order to prevent and reduce damage caused by deep-seated landslides, therefore, intangible measures including preparation of hazard maps as well as tangible structural measures as "passive mode" to protect roads and assets from direct hit by collapsed mass and debris flow have been studied and developed so far.

In other words, tangible structural measures to prevent deep-seated landslide at its source have not yet been specifically examined. This means, in Japan, tangible structural measures to control activity of gravity defamation, which is a precursor of deep-seated landslide, is yet examined either.





* Counting of Torrents: Even if multiple deep-seated landslides occurred in the same torrents, it is counted as 1onetorrent.

Figure 1-5 Risk Evaluation of Deep-seated Landslides of Torrents in the Kii Mountains and Locations of Deep-seated Landslide Caused by Typhoon No. 12

Source: MLIT of Japan https://www.mlit.go.jp/common/000223657.pdf

1.1.2 Gravity Deformation / Deep-seated Landslide in India (Paglajhora Landslide in West Bengal)

National Highway NH55 in West Bengal, India, connects the famous tourist destination of Darjeeling with Siliguri, the main city of West Bengal. NH55 climbs up a height of around 1,400 meters within a stretch of 40 km from Sukhna to Kurseong. A famous sinking zone of Paglajhora lies 4 km before Kurseong. The activities of Paglajhora Sinking Zone have decade's long history. Sujit Mandal summarized comprehensive review of the zone, in which subsidence of roads were reported.

The slope of Paglajhora in 2006 before the latest deep-seated landslide occurred is shown in Figure 1-6 (Left). The toe of the slope is located at a water colliding front of the river running along the toe. The slope just above the river, circled in the photo, seems slightly bulging.

In the photo taken in 2009 shown in Figure 1-6 (Center), multiple slope failures are aligned horizontally below NH55. The head scarps of such failures correspond to the convex knick line which runs about 150 meters below NH55. Just below NH55, sawtooth shaped scarplets, which is considered precursors of head scarps, branch downward from NH55 and develop nearly parallel to the convex knick line. The aforementioned slope with bulging remains intact along with substantial parts of covering vegetation.

In the photo taken in 2011 shown in Figure 1-6 (Right), the upper slope above the knick line completely collapsed and the current shape of the slope was formed. As seen in these photos, it is supposed that the collapse of the middle slope precedes the large-scale collapse of the upper slope of the knick line including NH55.

Geology of the Indian Himalayas is composed of 4 geological zones stretching east and west, among which Greater Himalayan Sequence underlies Paglajhora Sinking Zone. Greater Himalayan Sequence is composed of highly metamorphic rock of gneiss, migmatite, and quartzite. The slopes in Paglajhora Sinking Zone are very steep with average gradients of 60 ~70 degrees. Despite weak planes in highly metamorphosed rock, strong hardness of the bedrock maintains such steep slopes.

Figure 1-6 Overview image of Paglajhora

Source: Edited from Google Earth

Figure 1-7 shows the results of map reading of Paglajhora Sinking Zone and the surrounding slopes. A number of contour anomalies indicating sliding blocks of various sizes are

interpreted. Considering the steepness of the slope, most of the sliding blocks interpreted through map reading are considered as gravity deformation. The direction of the movement of each block almost coincides with the steepest direction of the slopes. Considering the said situation, creep deformation of bedrock slope is developing at such blocks. The creep deformation is facilitated by loosened zones in the rock slopes due to stress release, rather than constraints from geological structure.

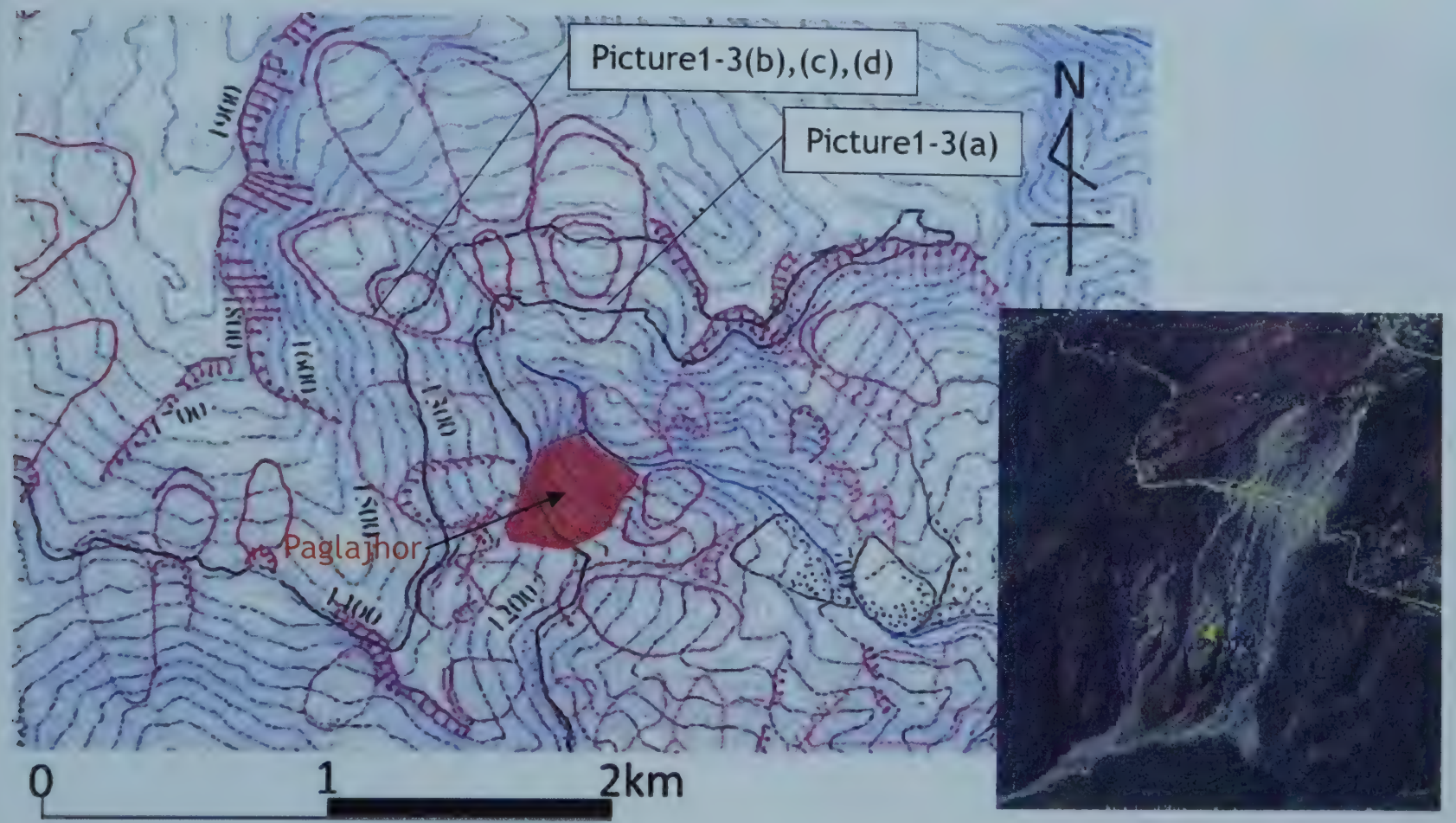


Figure 1-7 Google Earth bird's-eye view of Paglajhora and topographic interpretation results

Source: Left: JICA Project Team, Left: Edited from Google Earth

During the survey conducted in February 2020, on the slopes where the deep-seated landslide occurred, signs of bulging of slopes and sinking of the road as shown in Figure 1-8 were still observed in the section of more than 100 meters long. Along the track of the Darjeeling Himalayan Railway (DHR), which runs parallel to NH55 in Pagrajhora Sinking Zone, many traces of repeated repairs were observed as well. (Picture 1-2).

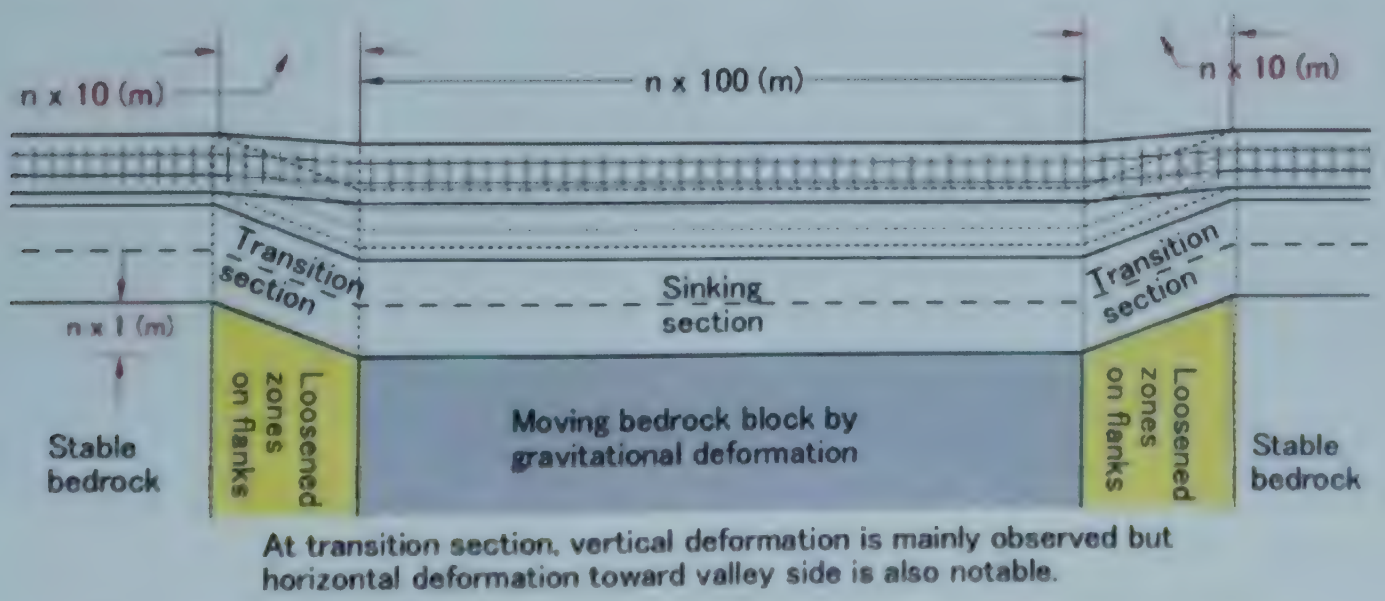


Figure 1-8 Typical Form of Road Sinking in Paglajhora (no scale)

Source: Kawamura, (2020) Sinking of Hill Road Due to Gravitational Deformation and Rate of Sinking at Paglajhora Sinking Zone in Darjeeling Himalaya in India

(a) overall view, (b) situation of the right side wall* taken from outside the deformation area, (c) situation of the left side wall* taken from outside the deformation area, (d) sinking phenomena in the left side wall, (e) abandoned old track observed in the left side wall, (f) train passing through the sinking area in the left side wall

Picture 1-2 Sinking condition of Paglajhora

Source: JICA Project Team

Picture 1-3 shows the sections where a series of the sinking of the road were confirmed in the Paglajhora Sinking Zone and surrounding slopes. A series of road sinking as shown in Picture 1-3 (a) were confirmed at all blocks of gravity deformation traversed by NH55 and interpreted by map reading shown in Figure 1-7. According to Kawamura et al. (2019) and Kawamura (2020), as shown in Figure 1-8, sinking sections often have transition sections on either side or both sides of a gravity deformation block at the boundaries with neighboring stable sections.

While a gravity deformation block develops, creep deformation proceeds in and around structurally weak planes of bedrock, which often form an asymmetrical wedge shape with various directions and dips. Thus, in some cases, a slip surface / creep zone of gravity deformation block may be formed along such asymmetrical wedge shape. An asymmetrical wedge shaped slip surface governs a dip of slip surface, which is often unrelated to the gradient of the slope surface. When a dip of slip surface is equal or gentler than a gradient of slope surface, subsidence at the boundary becomes unclear. This is why the transition section lacks on either side of a gravity deformation block

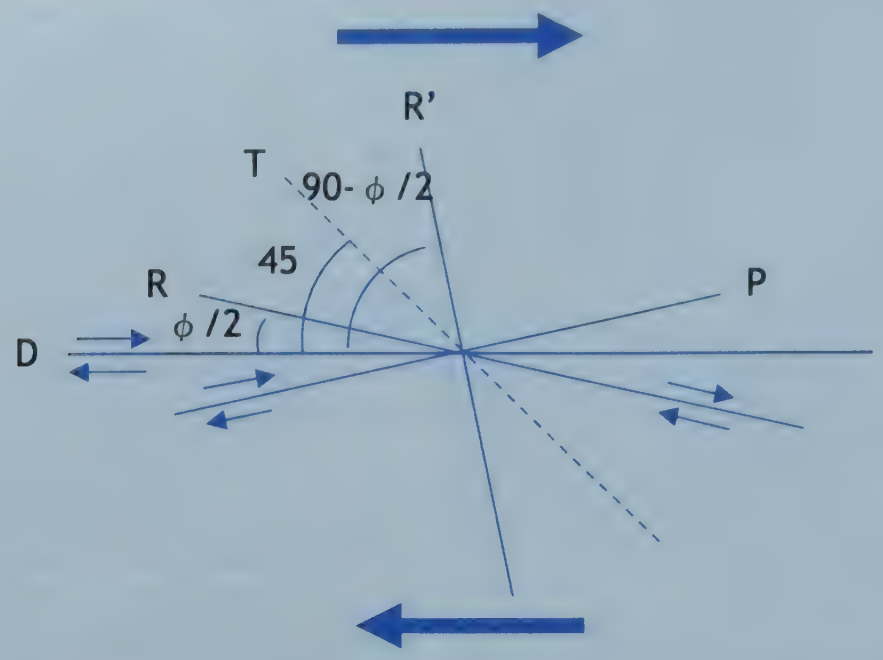
In Kawamura (2020), it is pointed out that the rail track of DHR has been repeatedly repaired to fill up the sinking in order to keep an operational gradient for trains, while NH55 tends to be left unrepaired as long as vehicles can travel through the steep transition sections.

Picture 1-3 (b), (c), and (d) show shear zones in a loosened zone of rock mass on the right flank of a developing gravity deformation block. Riedel shear structures are clearly observed in Picture 1-3 (c). The weak planes of surrounding bedrock have the following tendency; bedding plane of strike N70W and dip 46N and two directions of the schistosity with strike N68W and dip 58S as well as strike N44E and dip 82S. The gravity deformation blocks shown in Picture 1-3 (b), (c), and (d) have a basic structure with steep joint planes in the direction of the steepest gradient of slope, while in the transverse direction, the dominant schistosity forms the right frank and the conjugated joint planes forms the left flank.

(a) sinking road surface due to gravity deformation, (b) rock mass discontinuity observed on the right side wall of a gravitational deformation block, (c) Riedel shear surface group at loosening deformation zone, (d) displacement caused by rock mass deformation

Picture 1-3 Slope condition around the Paglajhora Sinking Zone

Source: JICA Project Team



D: principal displacement shear, P: thrust shear, R: Riedel shear, R': conjugate Riedel shear,
T: tension fracture, Φ internal friction angle

Figure 1-9 Technical Terms Related to Simple Shear

Source: JICA Project Team

1.2 Progress and Perspective of Studies on Deep-seated Gravitational Slope Deformation

1.2.1 Studies on Terrain of Deep-seated Gravitational Slope Deformation

(1) Terminology for Terrain of Deep-seated Gravitational Slope Deformation

Kojima (2018) comprehensively reviewed the studies on terrain of the deep-seated gravitational slope deformation (hereinafter referred as DSGSD) in Japan, and describes the perspective for the future studies. The terminology related to DSGSD are outlined below referring to the Kojima's paper.

In the Kojima's paper, the terms related to DSGSD were extracted from the various technical terms related to mass movement or slope movement caused by the gravity. These terms were then classified into three categories; terms related to terrain, movement, and geological phenomena, with overviews added by Kojima.

(i) Terms Related to Movement

Bedrock creep: A phenomenon in which bedrock gradually deforms and then collapses (near the slope surface) without a continuous slip surface due to the action by the gravity. "Rock mass creep" is used in the same meaning.

Sagging: A phenomenon in which a slope with sheared failure at the upper part shows displacement with hanging down in the lower part.

DSGSD: Movement in which a hill slope is gravitationally deformed involving deeper parts of the slope. This term is frequently used internationally in recent years. While the term "rock mass creep" or "gravity deformation" is used for continuous deformation of rock not only in deeper parts but close to the slope surface, "DSGSD" is a dedicated expression for deformation at deeper parts of the slope.

(ii) Terms Related to Geological Phenomena

Gravity fault: A term defined by the geomorphic agent. Gravity fault is formed by the gravity and thus is classified as non-tectonic fault. It is sometimes called a gravitational non-tectonic fault. Most of the gravity faults with the geomorphological features of DSGSD are normal faults.

(iii) Terms Related to Terrain

Double (multiple) ridges: Terrain with double or multiple ridges branching or running in parallel. Sometimes another ridge appears parallel to the main ridgeline. In other cases, when a certain ridgeline is traced, it is gradually getting unclear, then another ridgeline appears in parallel and gradually become clear to form the main ridgeline.

Linear depression: A linearly continuous depression formed between double or multiple ridges. It is also defined as a continuous depression formed between uphill-facing scarp and upper hill slope.

Ridge-top depression: One type of linear depression. This term is focused on the place where it develops.

Uphill-facing small scarp or scarplet: This term is defined as small cliffs formed on hill slopes and declining toward ridgeline / upper slope. In recent years, in the field of geomorphology, low scarp is sometimes used instead of small scarp.

Ridge-facing small scarp or scarplet: Same as uphill-facing small scarp or scarplet

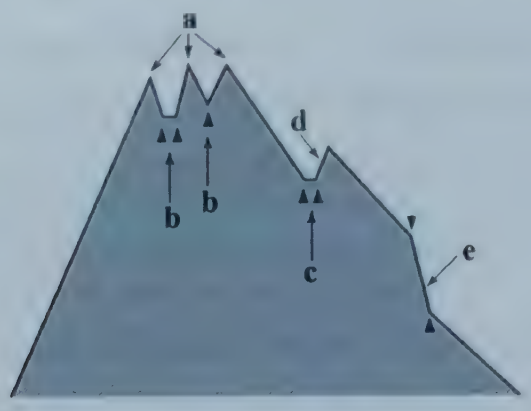
Reverse small scarp or scarplet: Same as uphill-facing small scarp or scarplet

Downhill-facing small scarp or scarplet: This term is defined as steep small/low cliffs formed on gentle hill slopes and declining steeply downward / valley side.

Scarplet: A collective representation of uphill-facing scarp or scarplet and downhill-facing scarp or scarplet.

Bulging: Convex deformation of hill slope.

Initial terrain of slope movement: A term that includes all aforementioned micro-topography, as an expression that regards DSGSD as a specific stage in the process of development of a deep-seated catastrophic landslide.



a: double (multiple) ridges; b: linear (ridge-top) depression; c: linear depression;
d: uphill-facing scarp or scarplet; e: downhill-facing scarp or scarplet

Figure 1-10 Classification and Technical Term of Topographic Features of DSGSD

Kojima (2018), recent progress and future perspective of research on deep-seated gravitational slope deformation in Japan from the engineering geological viewpoint, Jour. Geol. Soc. Japan, Vol.124, No.11

(2) Formation Factors of DSGSD

Different from landslides with landslide clay on gentle slopes, which are repeatedly activated and governed by shear properties of landslide clay on slip surface, the materials related to the movement / deformation of DSGSD are of fractured zones / parts of bedrock. Shear properties of fractured zones/parts of bedrock govern DSGSDs, and, because of this feature, DSGSD occurs only on steep rock slope. This filed of occurrence is one of prominent features of DSGSD.

As mentioned in the previous section, based on previous research results (e.g., Beck (1968)), in the areas where soft rock is distributed, such as tertiary sedimentary rock, landslides with landslide clay are often identified. On the other hand, however, DSGSDs are rarely found on natural slopes in such areas, where hill slopes tend to be gentle.

Actually DSGSD is generally found in various types of terrain from high mountains to low hills, but its density and development are heavily biased. Many researchers including as Yagi (1981), Matsuoka (1985), and Iwamatsu (1986) pointed out that geomorphological and geological factors cause such bias. Kojima (2018) comprehensively summarized distribution of DSGSD as follows:

>> DSGSD are predominantly identified in hilly areas with large undulations.
>> DSGSD develops on ridges and hill slopes at certain relative heights from bottom of valleys
>> DSGSD develops parallel to main ridges.
>> DSGSD often develops on gentle slopes above knick lines.
>> DSGSD often develops on the slope with bedrock with planar structures, such as shale and crystalline schist.
>> DSGSD often develops on the slope where a strike of planar structure of bedrock is parallel with direction of a ridge.
>> DSGSD sometimes develops even in massive rock under certain conditions; where lower slope frequently collapse or tensile strength of rock is reduced due to weathering, DSGSD can develop in massive granite.

In addition, Daimaru (2011) reported new cracks emerged at existing linear depressions in Iwate-Miyagi Nairiku Earthquake (2008) and discussed development process of scarplets in relation to seismic motion.

(3) Perspectives of Researches on Gravitationally Deformed Terrain

To clarify the cause and mechanism relating DSGSD development need further study. For example, other than formed by DSGLS, as pointed out by Kojima (2018), double (multiple) ridges can be formed by differential erosion where the strata with different resistance against erosion alternate on steep slopes. Other factors than DSGSD may form lineaments, such as faults, joints, and dikes on slope surface. The lineaments formed by the DSGSD should, therefore, be properly distinguished from those formed by other factors.

It should also be noted that detection of DSGSD does not directly mean the risk of enormous hazards such as deep-seated catastrophic landslides. Kojima et al. (2015) reported that the sediments found in some linear depressions had been formed for around 100,000 years. Kojima (2018) shows his concerns over excessive response against DSGSD considering it as the precursor of deep-seated catastrophic landslides. Kojima discussed DSGSD (c in Figure 1-10) on a gentle slope just above a knick line (a head scarp) can be considered as a precursor of a deep-seated catastrophic landslides, but DSGSD (b in Figure 1-10) on upper slopes or ridges far from knick lines should not be considered a precursor because deformation of such DSGSD has a long history of deformation term.

Remarkable progress has been made recently in spatial information technologies using satellites. Among them, the technology related to InSAR has a potential to realize a system for comprehensively and continuously monitoring micro deformation on slopes. Such technologies are expected to be extremely effective in extracting gravitationally deformed slopes with high risks of devastating deep-seated catastrophic landslides.

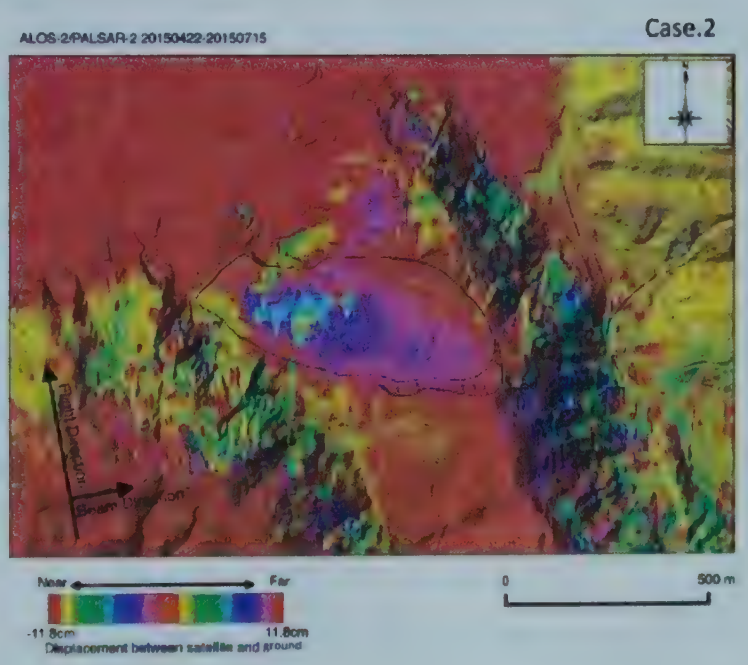


Figure 1-11 Extracting Landslide Area by InSAR Analysis

Source: Kito https://repository.kulib.kyoto-u.ac.jp/dspace/bitstream/2433/203224/1/27K-08-10.pdf

1.2.2 Progress and Perspective of Studies on Stability of Gravitationally Deformed Slopes

(1) Deformation and Fracture of Bedrock by Gravity

Chigira (1997) pointed out that scientific study on rock mass creep, in which bedrock gradually deforms due to the gravity while being fractured at subsurface, started quite recently. He discussed, therefore, that how rock mass creep structurally deforms bedrock and how rock properties changes are not well studied.

On the other hand, he discussed that promoting the studies on rock mass creep has three important meanings in the aspects of geology and engineering. The first is that rock mass creep is a phenomenon limited to the shallow part of the surface and properties and structures of bedrock are different from those in deeper part. Second, when rock mass creep progresses, it may cause landslides and slope failures. Third, bearing capacity and impermeability of bedrock are significantly reduced due to the fracture caused by rock mass creep.

From the aforementioned aspects, educating engineering geologists / civil engineers on rock mass creep has significant meaning, including characteristics of terrain and structure and properties of bedrock affected by rock mass creep as well as affects to slope stability.

Thus, the description of structural features formed by rock mass creep are summarized from Chigira (1997). The structure formed by rock mass creep greatly depends on planar structure in bedrock. If planar structure is well developed, the deformation by rock mass creep is basically caused by slide and tension fracture along the planar structure. As a result, folds and faults independent from tectonic force are formed.

The structure formed by such deformation roughly depends on geometrical relationship between planar structure and slope surface as a free face. Figure 1-12 shows various structure of rock mass creep focusing on the relationship between planar structure and slope surface. As shown in Figure 1-12 IV, when planar structure and slope surface are inclined in the same direction and the gradient of planar structure is gentler than that of the slope, in other words, in the case of a dip slope structure, shear failure occurs everywhere along the planar structure. As a result, rock mass fractures into flat plates, causing displacement between the flat plates, loosened zone is generated between the flat plates. As such deformation progresses, shear failures are concentrated to form fractured zones. At the initial stage, the fractured zones are formed locally. But it gradually extends across the entire slope, then rock mass creep turns to a rock slide.

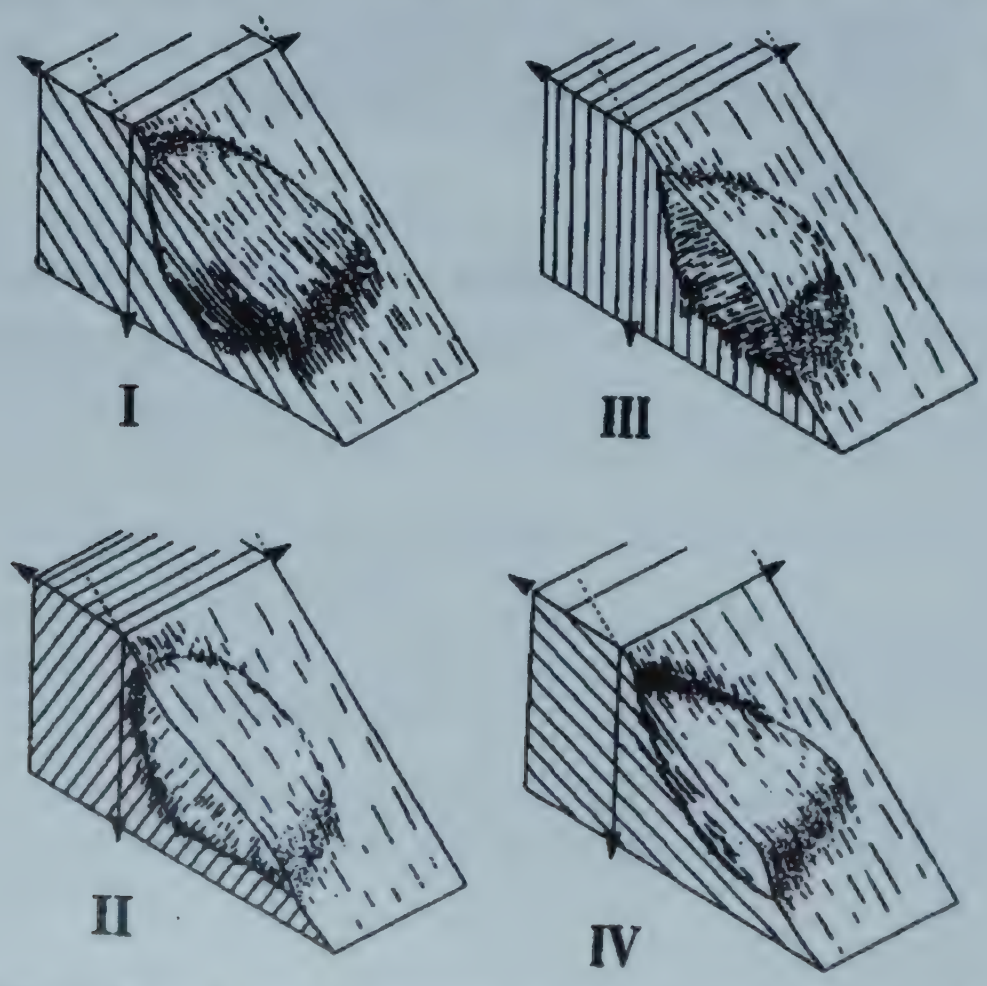


Figure 1-12 Rock Mass Creep in relation to Geological Plane and Slope Surface

Source: Chigira (1997)

As shown in Figure 1-12 I, when planar structure and slope are inclined in the same direction and gradient of planar structure is steeper, the force to induce shear failure is generated along the planar structure due to the gravity. The lower part of the planar structure is not released, however, so the buckling of flat plates occur.

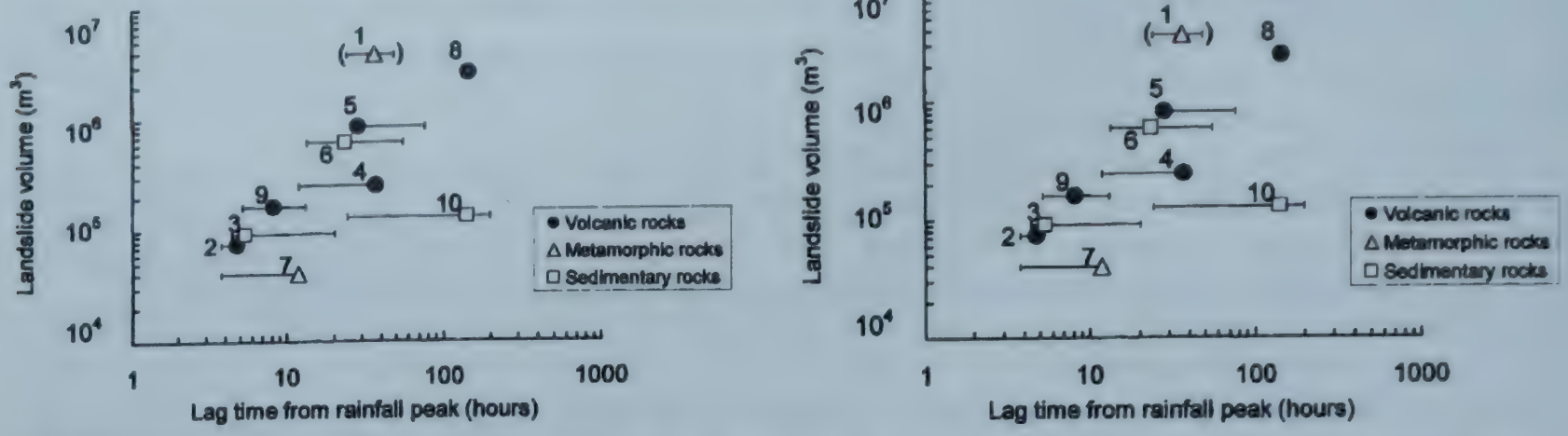
When the planar structure is almost vertical as in Figure 1-12 III, the strata topple towards the valley side. Such a phenomenon is called "toppling". In the initial stage, tensile fracture occurs due to bending of bedrock with platy structures. Then, with the progress of toppling, shear failure along the planar structure extends in bedrock, then the bedrock is fractured into flat plates.

For a series of gravitationally deformed terrain, Nishii et al. (2014) introduced identification of formation time of gravitationally deformed terrain and reenactment of long-term movement using surface exposure dating. The gravitationally deformed terrain targeted by Nishii et al. is the main ridgeline extending about 10 km in north-south direction from Mt. Eboshi-dake to Mt. Noguchigoro-dake located in the central part of the northern Japan Alps. At least 70 or more gravitationally deformed slopes are identified around the main ridgeline of granitic rock. As the dated age of multiple samples collected from the upper part of fracture surfaces are 0.9 to 9.1 ka, Nishii discussed these gravitationally deformed slopes are formed in Holocene. Comparing the periods when the gravitationally deformed slopes are formed and the relative heights of the cliffs formed by gravitational deformation, correlation between the periods of formation and the relative height of scarps is not recognized. On the other hand, many obtained values of exposed dating tend to show older age at the upper part of scarps and younger age at the lower, suggesting that these scarps were formed in multiple events rather than in one event.

Furthermore, Nishii et al. also examined the relations among factors related to formation of gravitational deformation, namely precipitation and seismic activity. Both may have influenced the beginning of gravitational deformation. Precipitation is considered to have increased significantly during the transition from the last glacial period to Holocene in the central northern Japan Alps. Nishii et al. discussed that, therefore, rapid down cutting of rivers and erosion of the lower slopes due to the increased precipitation probably induced gravitational deformation in the region. At the same time, among the records of past earthquakes in and around the studied area, Nishii et al. focused seismic events along the active faults of the northern part of Itoigawa-Shizuoka Tectonic Line. Nishii et al. concluded that the latest seismic event with a magnitude of M8 in 762 A.D. and the previous one almost coincide with the beginning of three gravitationally deformed slopes.

(2) Destabilization of Gravitationally Deformed Sloope Due to Precipitation

Hattanji (2003) points out that, although the collected cases are not yet sufficient, some deep-seated catastrophic landslides tend to occur hours or days after the peak rainfall. According to the relation between lag time and volume of landslides in Figure 1-13 (a) and the relation between lag time and maximum depth of landslides in Figure 1-13 (b), a clear positive correlation between the lag time and the scale of the landslides is understood. Hattanji supposed that the positive correlation means the deeper a slip surface runs, the longer infiltration of groundwater or the transmission of pressure to the slip surface takes time.



(a) Relation between collapsed soil volume and delay time (b) Relation between depth of collapse and delay time

* Each plot shows the lag time from the maximum rainfall peak of successive rainfall events, with the lag time range of the secondary peak shown as the error range.

Figure 1-13 Relation between Collapsed Size and Delay Time

Source: Hattanji (2003)

On the other hand, Imamori et al. (2016) studied the deep-seated catastrophic landslide in the Akatani of the Kii Mountains caused by the heavy rain by typhoon 1112 (Talas). And, the mechanism of the deep-seated catastrophic landslide was discussed from hydrological characteristics of the slope where the landslide occurred. After the disaster, 11 groundwater monitoring wells were installed around the landslide, and continuous monitoring were conducted. Based on these observation, Imamori et al. analyzed that the groundwater fluctuations before and after the rainfall of Typhoon 1511 (Nangka: total rainfall of 518 mm), which also caused heavy rainfall in the area in July 2015 (Figure 1-14 (a), (b)). The figure shows that the groundwater level rose about a height of 30 meters at A-2 and A-6. Imamori et al. pointed out the followings, therefore:

- The groundwater level around the head of the landslide rises in a short time due to heavy rain.
- Also, since the timing at which the water level rises differs for each monitoring well, existence of groundwater veins, through which groundwater flowing down into the landslide is inferred. The groundwater veins connect groundwater with the tributary torrents running at the upper slope, as shown by the arrows in Figure 1-14 (b).
- During the period without rainfall, the groundwater level of each hole is observed around the boundary between the loosened zone and the intact bedrock (40 to 60 m deep from the slope surface). And, from results of groundwater resistivity logging that penetrates the loosened zone, the groundwater supplied from the upper slope flows on the boundary as unconfined water in the loosened zone (namely, the creep zone in Figure 1-14(c)).

1.3 Deep-seated Landslide and Gravity Deformation

1.3.1 Investigation on Gravity Deformation to Detect Potential Deep-seated Landslides

(1) Triggers of Deep-seated Landslides

As mentioned in the previous sections gravity deformation is closely related to deep-seated landslides. In this section, firstly we discuss triggers of deep-seated landslide; heavy rainfall and earthquakes. Chigira (2015) summarized differences between deep-seated landslides caused by heavy rainfall and those by earthquakes, regarding geological structure which induced deep-seated landslides and types of gravity deformation on the slope before deep-seated landslides occurred.

Based on the studies of deep-seated landslides caused by typhoon Talas in 2011 and typhoon Nabi in 2005, Chigira claims the deep-seated landslides due to heavy rainfall are preceded by gravity deformation, and thus such gravity deformation can be detected from topographic features on hill slopes. Furthermore, Chigira also insists that even internal structure of the slopes where gravity deformation develops can be interpreted from topographic features of such slopes, and, based on such findings, even though it is less numerical analysis, risk of deep-seated landslide due to heavy rainfall may be predicted to some extent.

Table 1-1 is a reorganized chart presented by Chigira. The risk levels shown here are defined as follows:

I: Slope is deformed by gravity, however risk level of deep-seated landslide is low.
II: Risk level of deep-seated landslide is moderate.
III: Risk level of deep-seated landslide is high.
IV: Risk level of deep-seated landslide is very high.

As for the earthquake-induced deep-seated landslides, based on the studies of Off the Pacific coast of Tohoku Earthquake in 2011, the Sumatra Earthquakes in 2009, the Wenchuan Earthquake in 2008, the Iwate-Miyagi Nairiku Earthquake in 2008, the Kashmir Earthquake in 2005, and the Mid Niigata Prefecture Earthquake in 2004, Chigira reported that substantial numbers of deep-seated landslides occurred at the slopes where chemical weathering or gravity deformation was so extremely developed that the slope was about to collapse. Accordingly, by extracting such slopes, he suggests that deep-seated landslides by earthquake can be predicted as well.

Matushi et al. (2012) focused on heavy rainfall as well. From August 31st to September 4th, 2011, Typhoon Talas in 2011 provided heavy rainfall of up to 1,800 mm on the Kii Peninsula and induced substantial numbers of deep-seated landslides in the central part of the peninsula from the afternoon of September 3rd to 4th. Matsushi summarized the relationship between the rainfall history and the fields where such deep-seated landslides occurred and discussed an interesting viewpoint.

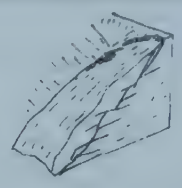
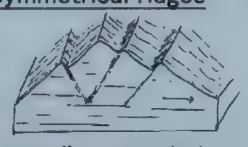
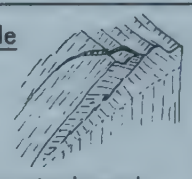
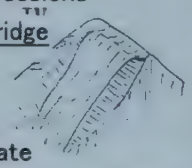
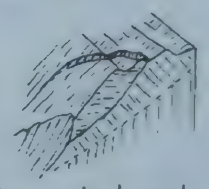
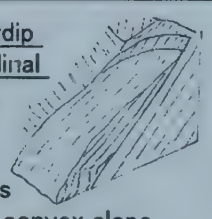
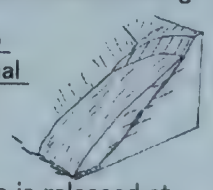
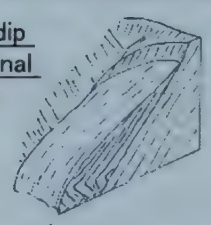
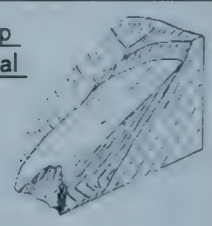
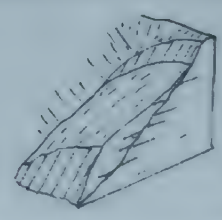
Figure 1-16 shows the characteristics of the rainfall and the sites of deep-seated landslides when Typhoon Talas struck the Kii Peninsula in 2011. The figure shows the center of total rainfall distribution as well as the center of point density of the deep-seated landslides. Interestingly those centers are deviated. The center of precipitation is located on the northeast side of the Omine Mountains. Matsushi considered the rainfall by the typhoon was the orographic one, generated by the southeasterly winds of the typhoon blocked by the Kii Mountains.

On the other hand, deep-seated landslides occurred mainly on the west side of the Omine Mountains, especially in the upper reaches of the Totsugawa River, which is a tributary of the Kumano River. As shown in Figure 1-16 (a), however, there is no remarkable difference in the change with time of rainfall at the Kazeya AMeDAS station near the area where many deep-seated landslides occurred and the Kamikitayama AMeDAS station where only few deep-seated landslides occurred. Furthermore, the total rainfall was rather higher at the Kamikitayama station, where the rainfall intensity was generally higher.

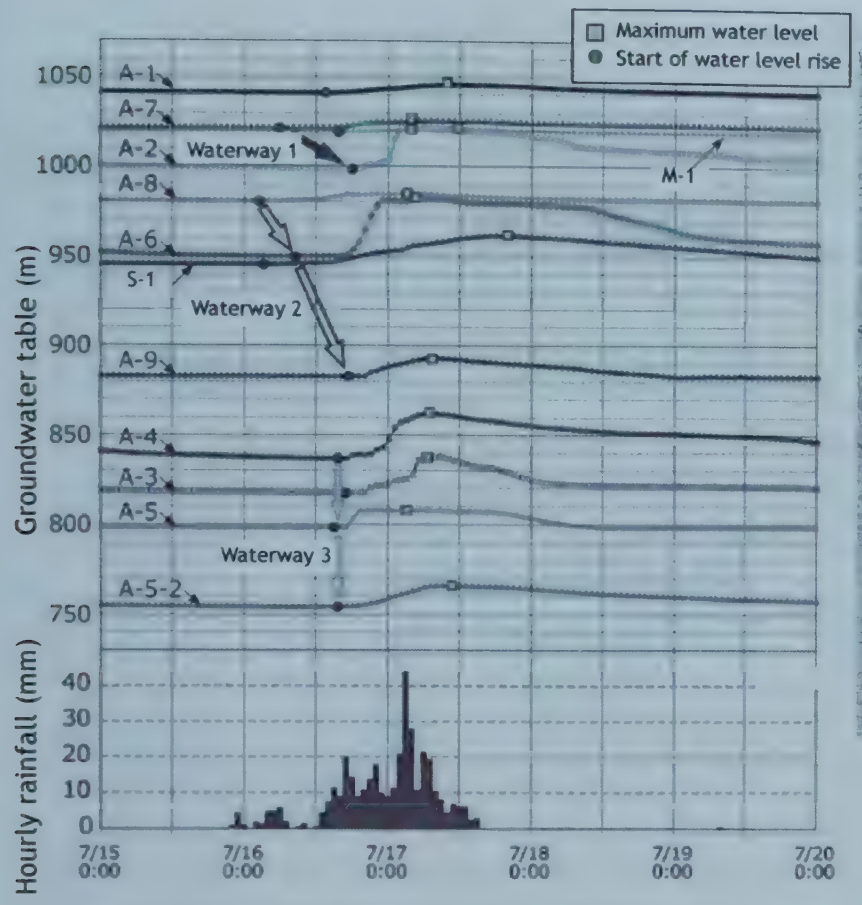
From the above discussion, Matsushi insists that deep-seated landslides are governed by many factors including intensity of trigger rainfall. Considering that large-scale convex knick

lines are recognized on the slope in the upper reaches of the Totsugawa River (Hiraishi and Chigira, 2011), Matsushi supposes that landform evolution and long-term rainfall history may affects the fields of occurrence of the deep-seated landslides by the thphoon

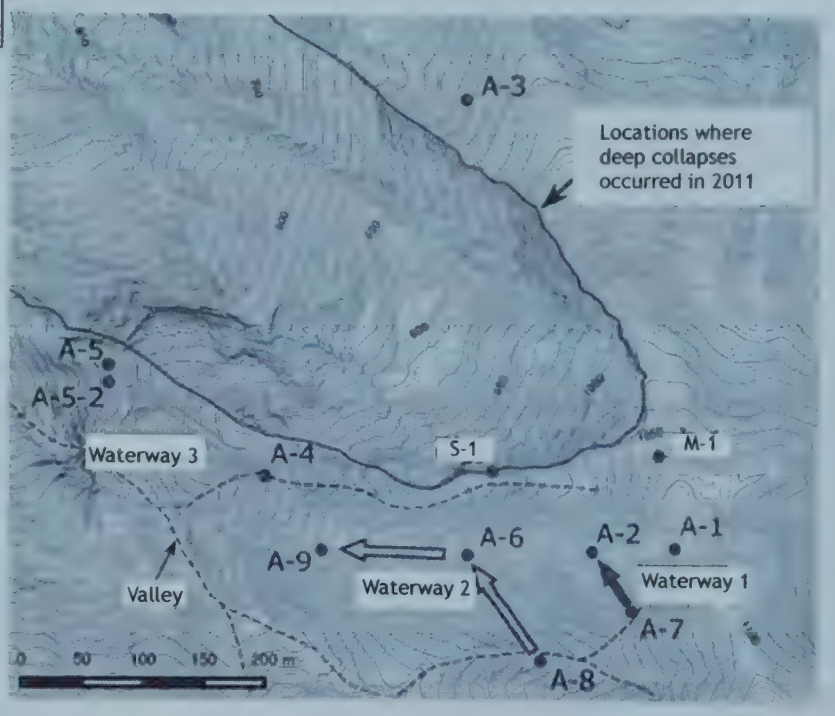
Table 1-1 Types of Gravity Deformation and Risk of Deep-seated Landslide by Rainfall

Types of Slope Deformation	Risk level I	Risk level II	Risk level III	Risk level IV
Irregularly undulating slopes	The slope is irregularly undulating without any valleys	 Discontinuous echelon scarplets are formed	Small scarplets continue to the gullies of the both flanks. The toe of the released slope has not been eroded.	The scarplets are continuous. The toe of the slope is collapsed.
Linear depressions	Symmetrical ridges  Laterally expanded One side ridge Only uphill–facing scarplets and linear depressions		One side ridge  The crescent–shaped downhill–facing scarplet is formed crossing the uphill–facing scarplets and linear depressions Top of a ridge  Any scarplets that elongate from the top of the ridge to the bottom do not exist	One side ridge  Crescent–shaped downhill–facing scarplet that is crossing uphill–facing scarplets and linear depressions, with collapses at the toe
Large head scarps or ridge-top depressions		Underdip cataclinal slope  Scarps and a convex slope, with weak buckling Ridge–top depressions crossing the ridge, with weak buckling Overdip cataclinal slope  Stress is released at lower part of the slope	Underdip cataclinal slope  Scarps and a convex slope, with strong buckling	Underdip cataclinal slope  Scarps and a convex slope, with strong buckling and collapses at the toe Overdip cataclinal slope  Collapses at toe of the slope

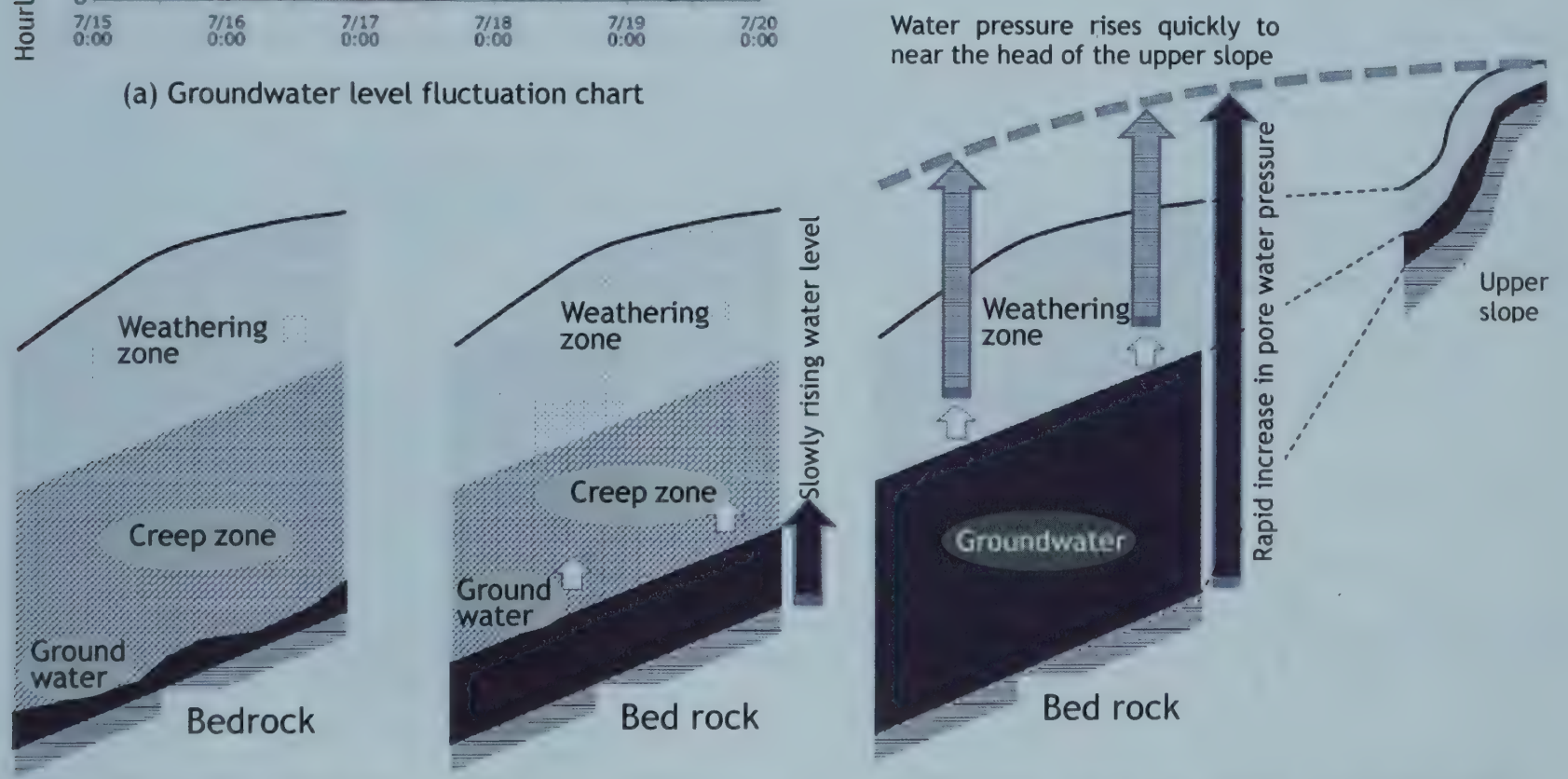
Source: modified from Chigira (2015)



(a) Groundwater level fluctuation chart



(b) Borehole layout



Normal (unsaturated in creep zone) Unconfined groundwater flows through the creep zone

During rainfall (unsaturated in creep zone). Slow rise of water level in unconfined groundwater

Extreme rainfall events (saturation in the creep zone) As soon as the creep zone is saturated, the groundwater becomes pressurized

(c) Pressure generate mechanism of groundwater in creep zones

Figure 1-14 Behavior of Pressurized Groundwater in Creep Zones

Source: Imamori et al. (2016)

Unconfined groundwater does not show a significant rise in water level if amount of rainfall is not sufficient. This condition facilitates weathering in the shallower part of the slope. Due to weathering or decomposition of rock, clay or fine particles increase in the weathering zone from the slope surface to the depth of several tens of meters. These facts indicate that the permeability of the weathering zone is significantly lower than that of the creeping zone.

Based on these field observations, Imamori et al. proposed a process as shown in Figure 1-14 (c) as a mechanism of triggering a deep-seated catastrophic landslide:

i) Ordinary, deep groundwater supplied from the upper slope flows around the bottom of the creep zone as unconfined condition.
ii) During rainfall, the amount of groundwater flowing into the bottom of the creep zone increases. The rise in unconfined groundwater level is relatively slow in the creep zone

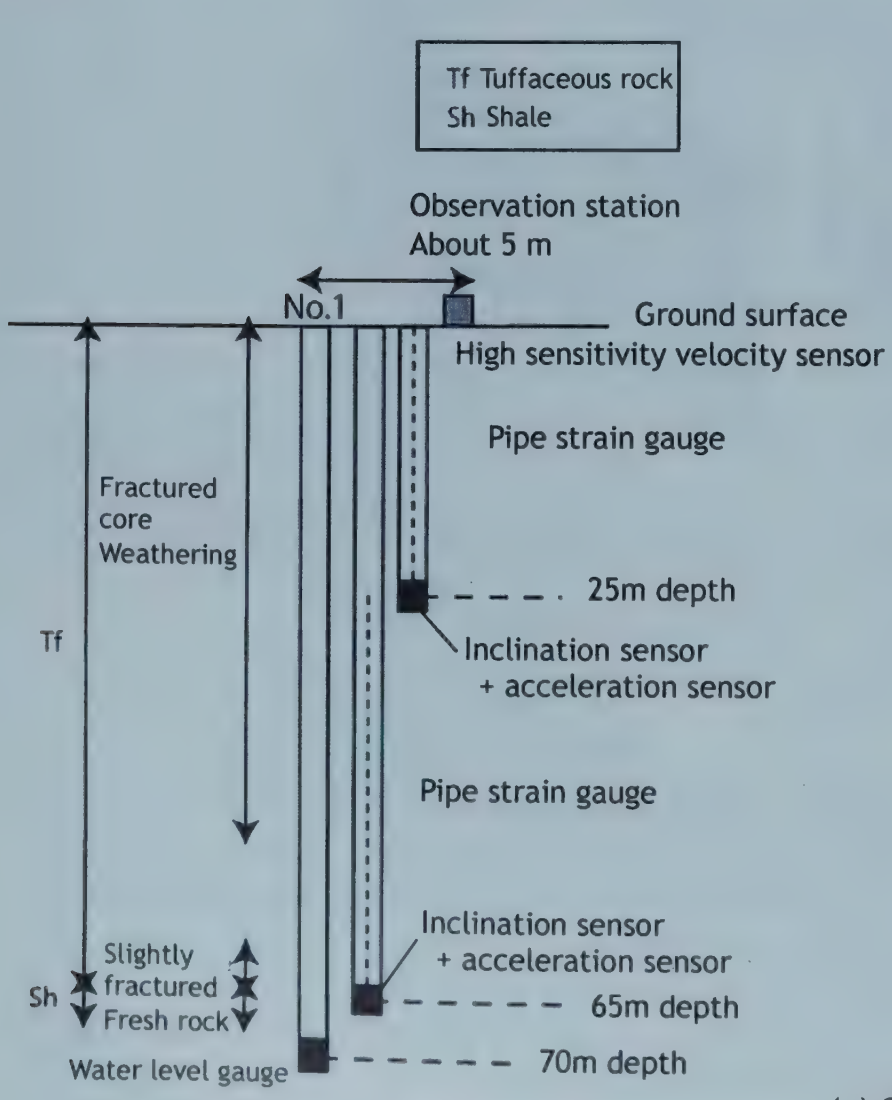
where bedrock is fractured remarkably, however.

iii) During abnormally heavy rain, the weathered zone above the creep zone acts as the upper constraining floor with a cap-rock structure. The creep zone is saturated and, due to the supply of water from upper slope, ground water in the creep zone turns to confined condition. By being connected to the surface / ground water of the upper slope through groundwater veins, the pore water pressure in the creep zone drastically increases up to the hydraulic head of unconfined groundwater table formed in the upper slope.

(3) Gravity deformation due to earthquake Destabilization of slope

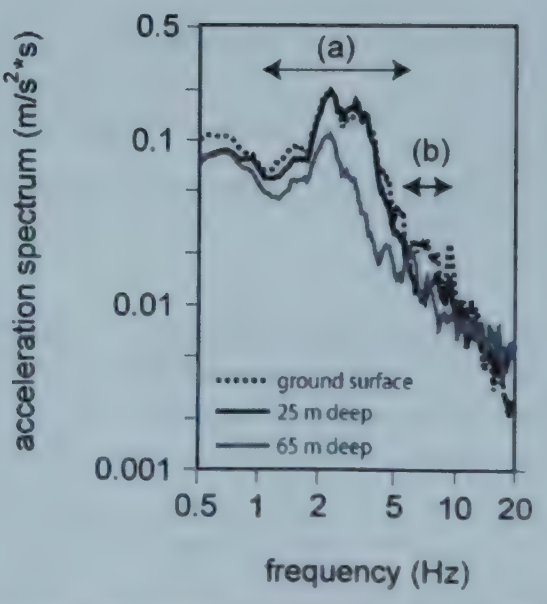
It is generally understood that deep-seated catastrophic landslides are induced by earthquakes or heavy rains. Sufficient research has not been conducted on the relation between gravity deformation and these triggers, however.

Doi et al. (2017) applied water and pressure-proofed integrated sensor of accelerometer and inclinometers and installed at 2 points of different depth of one borehole on a gravitationally deformed slope in the Kii Peninsula. Doi et al. observed these sensors continuously. The purpose of the study was to detect movement of loosened zone and accompanied deformation of a gravitationally deformed slope during an earthquake. As a result, an earthquake with magnitude 6.5 with an epicenter in the source region of the Tonankai-earthquake was successfully recorded. After analyzing seismic acceleration wave records, it was found that the amplitudes in the frequency band of 1-6Hz are 2-7 times larger in the gravitationally deformed block (at 25m) than those in the fixed point (at 65m) in bedrock. Rock mass creep is predominantly observed from the surface as a free face to shallow parts of the ground up to a certain depth. Thus, it is easily understood that gravitational deformed process is accelerated by vibration of earthquakes. And sometimes an earthquake may trigger a deep-seated catastrophic landslides.



(a) Sensor layout

(b) Newly developed sensor



(c) Observed horizontal acceleration spectra with instrumental corrections

Figure 1-15 Observed Seismic Behavior of Gravitationally Deformed Slope

Source: Doi et al. (2017)

Table 1-2 Relationship among Collapse by Earthquake, Geological Structure and Gravity Deformation

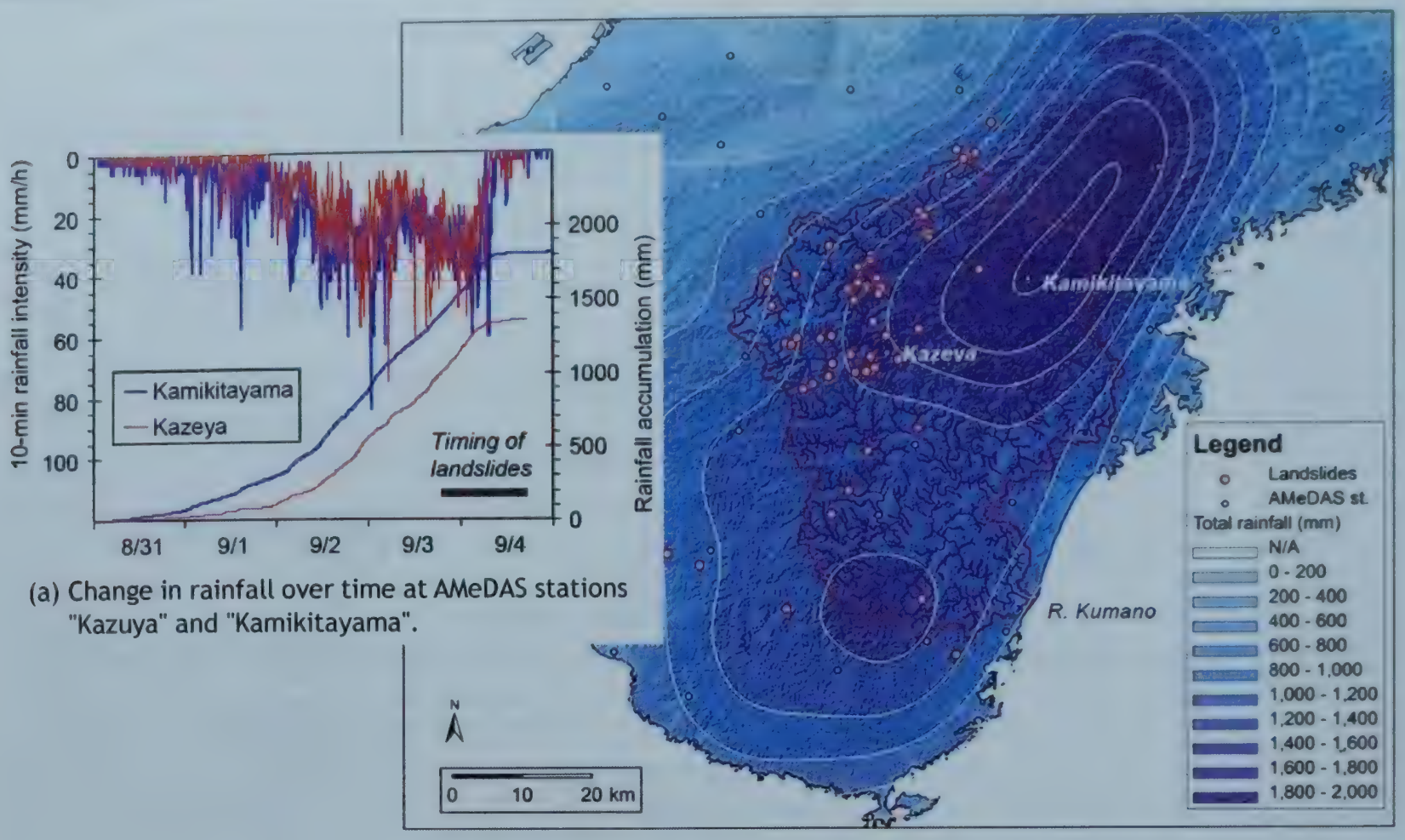
Earthquake	Country	Magnitude	Seismic intensity at the landslide site	Landslide	Volume (10^6m^3)
715 earthquake	Japan	M 6.5-	Unknown	Ikeguchi	93
1707 Hoei	Japan	M 8.4		Kanagi	85
1985 Papua New Guinea	Papua New Guinea	M 7.1	MM 8? (14 km from the epicenter)	Bairaman	200
1999 Chi-Chi	Taiwan	Mw 7.6	465.3 gal EW 370.5 gal NS, and 274.7 gal UD 6 km north of the site	Chiu-fen-erh-shan Tsaoling	50 125
2004 Mid Niigata Prefecture	Japan	Mw 6.6 (Mj 6.8)	6+～7 (JMA)	Higashitakezawa Shiono Terano Kazefukitoge	2 5 0.5 0.09
2005 Northern Pakistan	Pakistan	Mw 7.6	MM 8	Dandbeh Pir Bandiwala	65 1
2008 Wenchuan	China	Mw 7.6	824.1 gal EW, 802.7 gal NS, and 622.9 gal UD	Daguanbao Yinxinggou	837 1007
2008 Iwate Miyagi Inland	Japan	Mw 6.6 (Mj 6.8)	328 gal EW, 413 gal NS	Aratozawa	67

Earthquake	Rock type	dip (˚)	Structure*	Precursory landform	Reference
715 earthquake	Sandstone, mixed rocks, green stone	50-60	UC Bt	Head scarp	Chigira (2013)
1707 Hoei	Sandstone, mudstone	60-90	A FT	Furrows	Chigira (2000)
1985 Papua New Guinea	Limestone	8	OC U	Linear depression	King et al. (1989)
1999 Chi-Chi	Sandstone, mudstone, shale ditto	20-36 14	UC B OC U	Linear depression, Steps V-shaped linear depression	Wang et al. (2003) Chigira et al. (2003)
2004 Mid Niigata Prefecture	Sandstone, mudstone ditto ditto ditto	20 14 14 30-42	OC CU OC CU OC CU UC B	Head scarp Head scarp Head scar Unknown	Chigira and Yagi (2005)
2005 Northern Pakistan	Sandstone, mudstone Sandstone, mudstone	20 (plunge of a syncline) Unknown	OC CU Unknown CU	Small scarps Head scarp	Chigira (2007), Schneider (2008)
2008 Wenchuan	Carbonate rocks Carbonate rocks	35-38 (oblique) 25?	UC B OC U	Linear depression Unknown	Chigira (2010)
2008 Iwate Miyagi Inland	Sandstone, siltstone, tuff, welded tuff	0-2	OC CU	Linear depression	Ohno et al. (2010)

*; OC: overdip cataclinal; UC: underdip cataclinal; A: anaclinal: Bt: buttress; B: buckling: FT: flexural toppling; U: undercut; CU: collided the opposite slope then undercut

**; Usami (2003)

Source: Chigira (2015)



(a) Change in rainfall over time at AMeDAS stations "Kazuya" and "Kamikitayama".

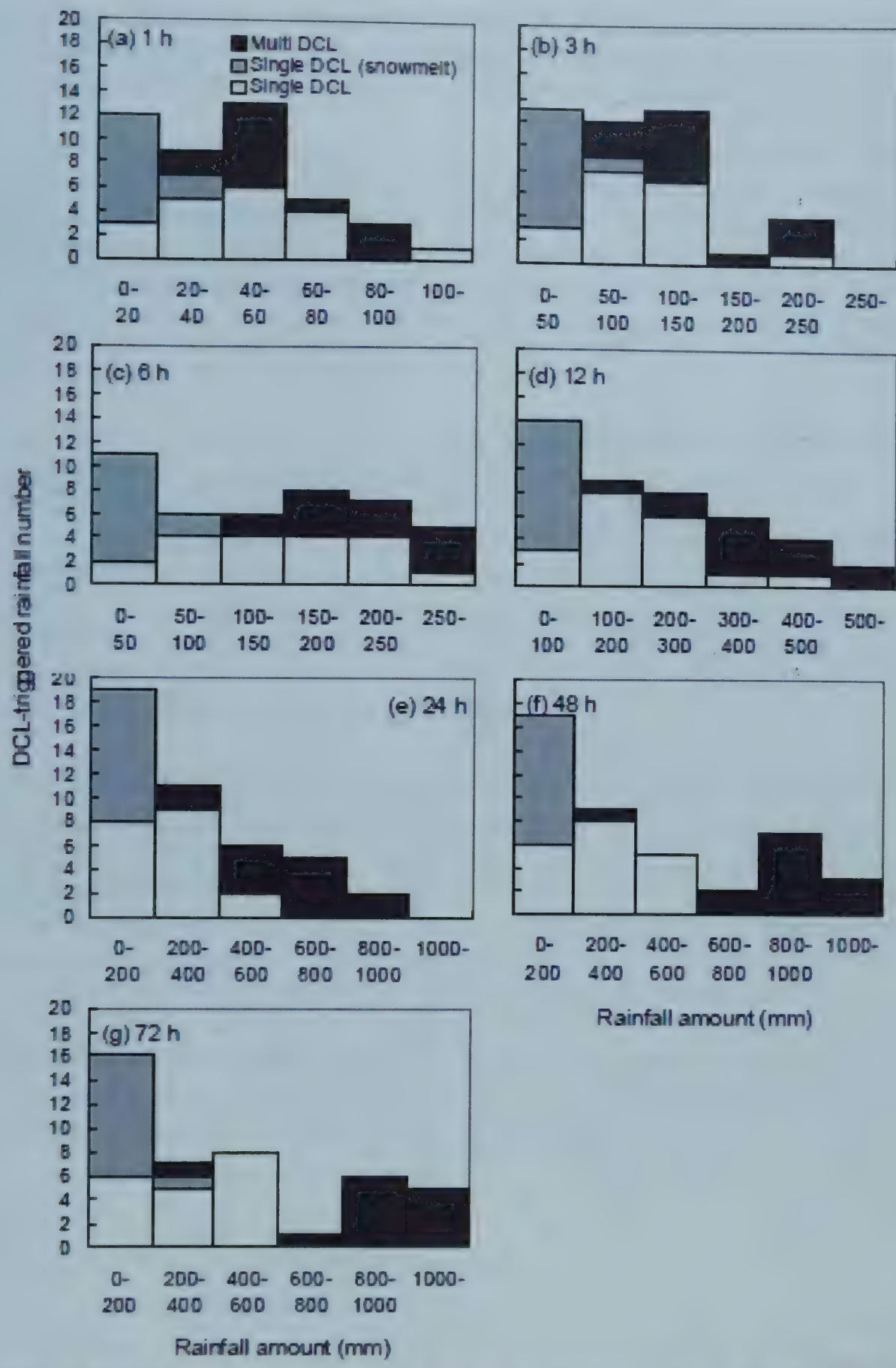
(b) Spatial distribution of total precipitation and deep-seated landslide due to Typhoon No.12 in Kii Peninsula, Japan in 2011

Figure 1-16 Rainfall Characteristics during Typhoon No. 12 in 2011 in the Kii Peninsula

Source: Matsushi (2012)

In addition, Uchida et al. (2012) conducted a survey on the rainfall when deep-seated landslides occurred in recent years and found that most of the rainfall that triggered multiple deep-seated landslides exceeded an accumulated precipitation of 600 mm in continuous 48 hours. The same tendency was also recognized in the Typhoon Talas in 2011. Figure 1-17 shows the frequency distribution of the maximum rainfall analyzed from several rainfall calculation periods of the rainfalls which induced deep-seated landslides. Figure 1-17 is interpreted as follows:

- When the rainfall calculation period is shorter than 12 hours, the maximum rainfall of the single events not affected by thaw and that of multiple events cannot be clearly separated.
- When the rainfall calculation period is 48 hours or longer, except one case of multiple event, the maximum rainfall of 12 multiple events is far larger than that of the single events; multiple and single events are separated clearly. The most of the multiple events were caused by rainfalls of over 600 mm in 48 hours.



Maximum rainfall in calculation periods of (a) 1h, (b) 3h, (c) 6h, (d) 12 h, (e) 24 h, (f) 48 h and (g) 72 h

Figure 1-17 Distribution Analysis of Deep-seated Catastrophic Landslide Triggered by Rainfall

Source: Uchida et al. (2012)

Based on site observation of deep-seated landslides and behavior of excess pore water pressure measured in laboratory tests, Kinoshita et al. (2013) discussed the dynamic mechanism on the slip surface of the multiple deep-seated landslides that occurred in the Kii Mountains in 2011.

Figure 1-18 shows the results of creep test of Nagatono landslide, one of the deep-seated landslides investigated by Kinoshita et al. The left figures show the behavior of amount of shear displacement, shear stress, normal stress, pore water pressure (P1, P2, P3: near the shear plane, P4: upper surface of the specimen) between the elapsed time of 21,000 seconds (before the displacement starts) and the elapsed time of 28,000 seconds (after the displacement starts). The pore water pressure P1 gradually generated negative pressure in the process of decreasing

normal stress just before the start of sudden displacement. Immediately after the shear displacement occurred, the pore water pressure gauges P1 and P2 measured negative pore water pressure of nearly 100 kPa, indicating that negative pressure was predominantly generated during the displacement shown in the figure.

The right figures show the behavior of shear stress and pore water pressure during the rotation of 1,080 degrees (3 rotations) after the start of displacement of the same test. In the figures, positive pressure gradually got dominant up to the rotation angle of around 200 degrees and positive excess pore water pressure were predominantly measured by all three pore water pressure gauges until the displacement exceeding 700 degrees. Furthermore, as the displacement progressed, the pore water pressure gauges P1 and P2 measured positive excess pore water pressure close to 300 kPa.

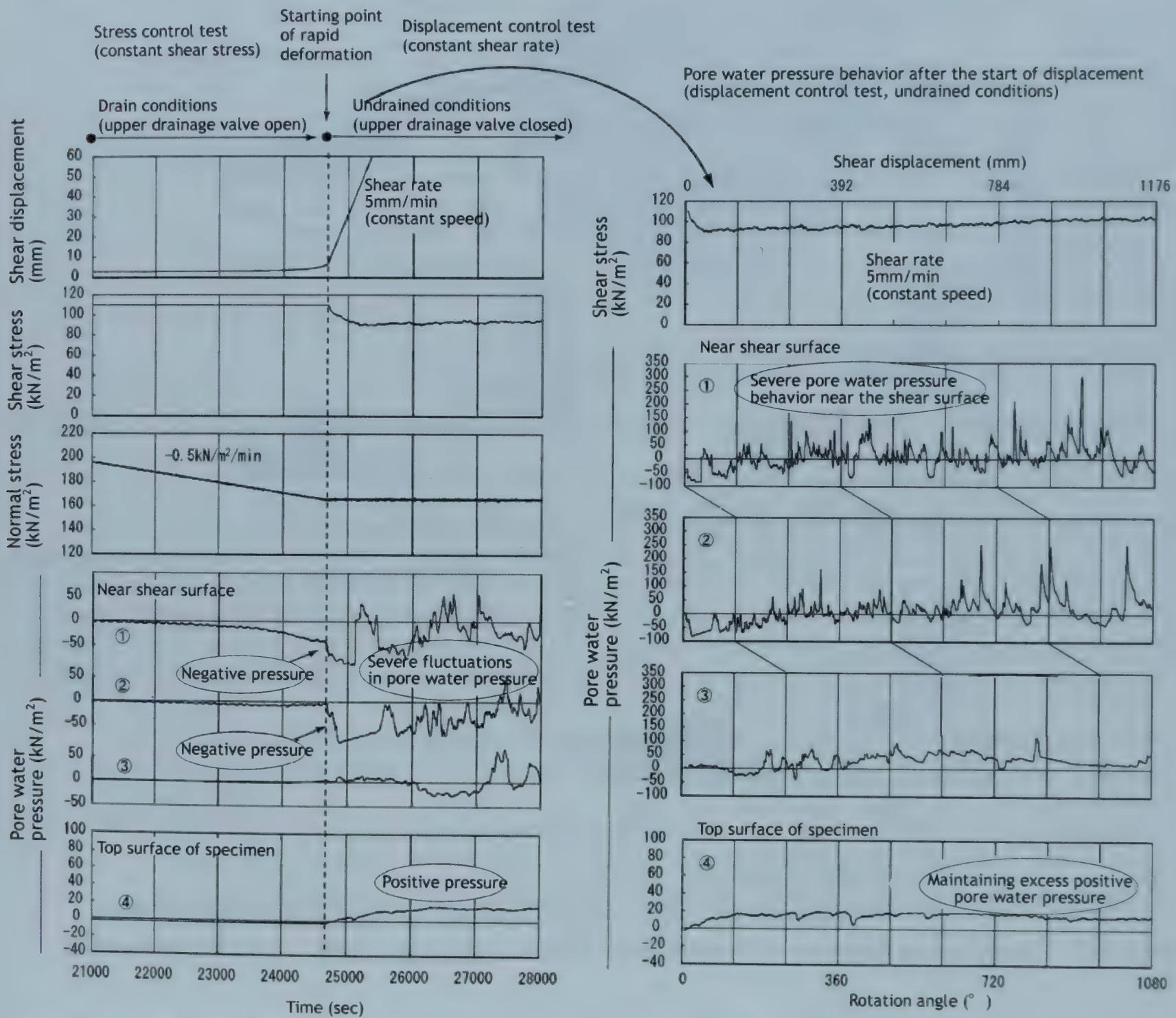


Figure 1-18 Results of Stress-controlled Ring Shear Test with Sample from Nagatono Landslide

Source: Kinoshita et al. (2013)

In addition to the laboratory test results, based on the results of field survey, Kinoshita claimed that, with high probability, open cracks and loosened zones near the slip surface were considered to be filled with groundwater. And Kinoshita also suggested that, before the deep-seated landslide occurred, the upper part of the original slope collapsed or the upper sliding block ran onto the head of the lower part, as shown in Figure 1-19, which Kinoshita used to explain the triggering mechanism of a deep-seated landslide.

In the "Manual on the method for extraction of torrents prone to a massive landslide occurrence (draft)", Tamura et al. (2008) stipulated the method of extraction of the torrents with high risk of deep-seated landslide from a whole targeted area based on the following three criteria:

1) Presence of ancient deep-seated catastrophic landslide traces.
2) Presence of geological conditions as well as micro-topographic features caused by long-lasting gravity deformation.
3) Presence of topographical indices.

In addition, Tamura proposes a weighing that the more a torrent falls under criteria of dangerous slope, the higher the risk of the torrents should be evaluated as shown in Figure 1-21.

By referring the research by Kinoshita et al. (2016) reported in Technical Note of PWRI No. 4333, the method promoted by MLIT of Japan for extracting the slope prone to deep-seated catastrophic landslide and estimating its risk is introduced below.

Process of extracting deep-seated landslide prone slopes are composed of "primary extraction (A-1)", "secondary extraction (A-2)", and "risk assessment of the slopes" (A-3). The primary extraction (A-1) is to extract slopes where deep-seated catastrophic landslide may occur. The secondary extraction (A-2) is to set priority for the following investigation. And the risk assessment of the slopes (A-3) is to assess the risk of and to estimate the scale of possible deep-seated catastrophic landslides on the extracted slopes.

A-1: Extraction of Slopes Where Deep-seated Catastrophic Landslide May Occur (Primary Extraction)

Deep-seated catastrophic landslides often occur on the slope close to or adjacent to the traces of old deep-seated landslides. Detecting the traces of old deep-seated landslides is, therefore, important. In addition, specific micro-topographic features are often formed on the slope where a deep-seated landslide occurs. Such micro-topographies landforms are considered as precursors and worth to extract. Digital elevation model (DEM) data produced with air-borne laser profiler, hereinafter referred to as laser profiler data, produces detailed topographic maps with detailed contour lines as well as post-processed thematic maps, such as slope gradation maps. Interpreting detailed contour maps and slope gradation maps as well as aerial photographs, traces of deep-seated catastrophic landslides and precursors of micro-topographic features are identified and, based on them, the slopes with risk of deep-seated catastrophic landslide are extracted.

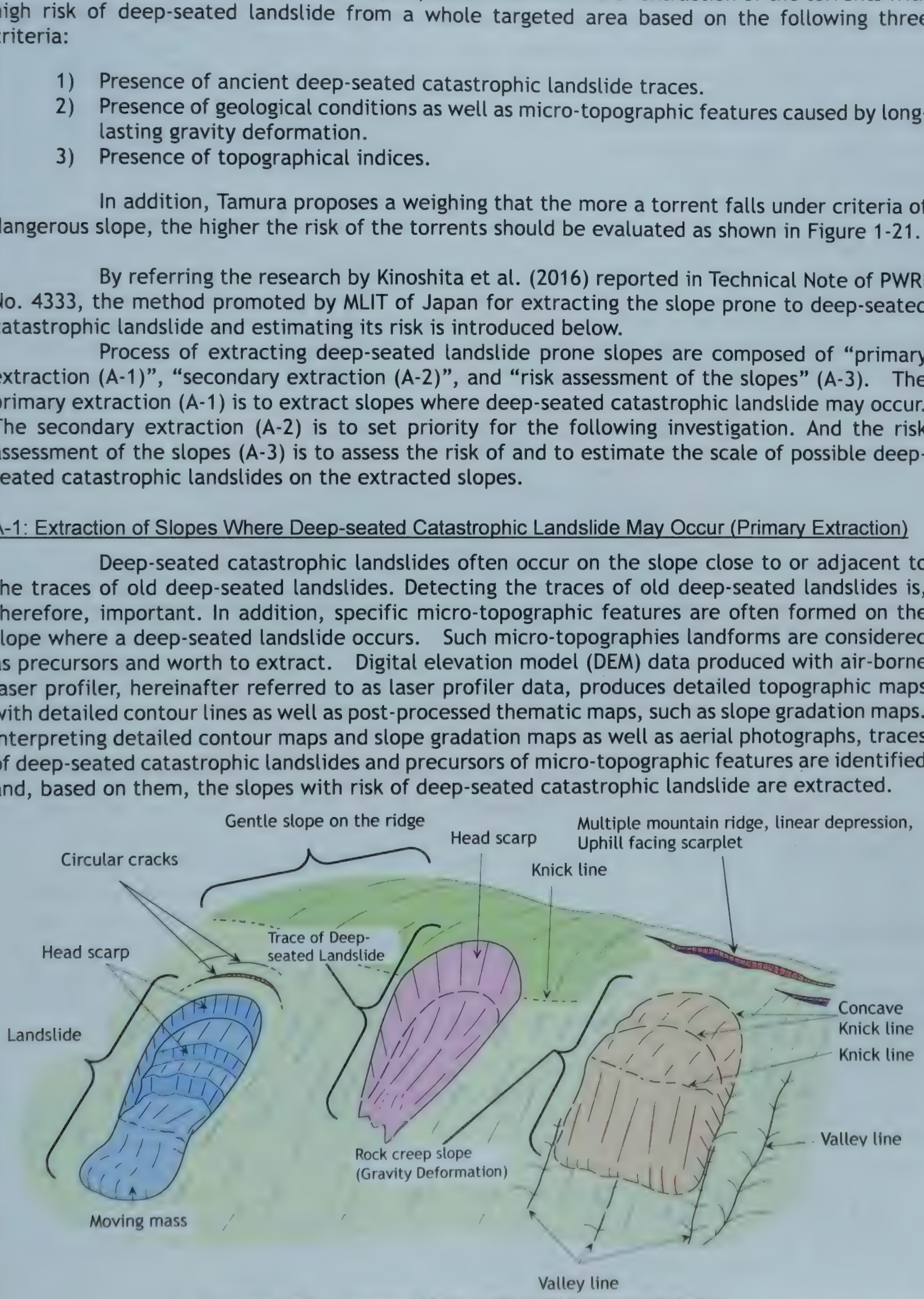


Figure 1-22 Image of Extracting Micro-topography

Source: Uchida et al. (2011) partially modified by Kinoshita et al. (2016)

A-2: Setting of Priority for Following Investigation (Secondary Extraction)

Based on the comprehensive consideration of social and topographical conditions, priority for the subsequent investigation for the slopes with risk of deep-seated catastrophic landslide are decided. Social conditions are set by organizing the locations of targets to protect and the importance of each facility, such as residential houses, public buildings, shelters for evacuation, etc.

Topographical conditions are set by performing topographical analysis using laser profiler data. By performing topographical analysis of targeted slopes, setting indices for topographical and geological conditions as well as numerical analysis with such indices are available, which enables quantitative assessment of the slopes.

The slopes with risk of deep-seated catastrophic landslides are highly likely to be affected by hydro-topographical conditions, such as gradient of natural slope and extent of catchment area. Gradient of natural slope and extent of catchment area organized by ratio of deep-seated landslide occurrence can be used as index threshold for hydro-topographic conditions.

On the other hand, since most of the traces of deep-seated landslides are accompanied with micro-topographical features related to gravity deformation, topographic factors relating to gravity deformation should be used as index threshold for topographic conditions.

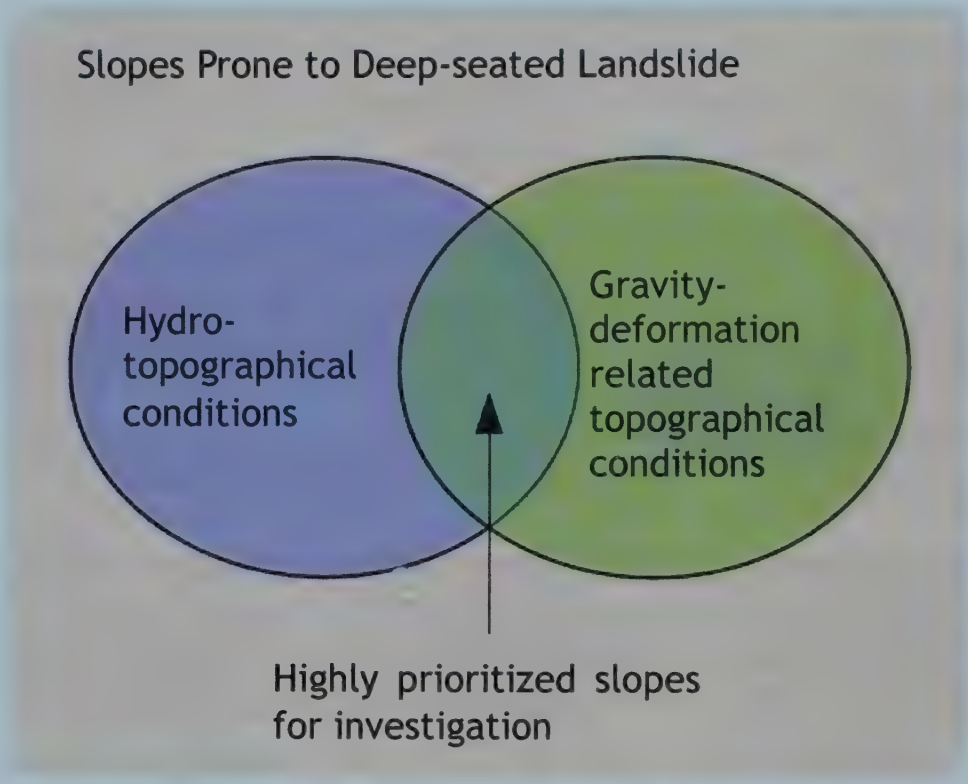


Figure 1-23 Image of Priority for Investigation Set by Topographic Conditions

Source: Kinoshita et al. (2016)

A-3: Risk Assessment of Slopes Where Deep-seated Catastrophic Landslides May Occur

Risk assessment are conducted for the slopes where deep-seated catastrophic landslides are anticipated based on the combination of the risk and scale of deep-seated landslides. The risk of the targeted slopes is assessed based on the following studies and investigations;

- Field investigations of topographical and geological conditions
- "Distortion rate" calculated from expansion rate of scarplets formed on the slope, and
- Hydrological survey including ion analysis to understand the peculiarity of slopes by comparing hydrological indices with the surroundings.

In hydrological surveys, electrical conductivity, water temperature, pH, quantity of spring water, flow rate, and air temperature are measured. Ion analysis includes 7 major dissolved ions (sodium ion Na^+, potassium ion K^+, calcium ion Ca^{2+}, magnesium ion Mg^{2+}, chloride ion Cl^-, sulfate ion SO_4^{2-}, hydrogen carbonate ion HCO_3^-) and silica.

With Figure 1-19, Kinoshita proposes the triggering process as follows:

- The upper block slides firstly and runs on the head of the lower block, causing a large compressive deformation on and slip surface of the lower block.
- An abnormal positive and negative excess pore water pressure are generated.
- Excess pore water pressure on the slip surface is further increased by sliding.
- Furthermore, fine particles generated by crushing during deformation/sliding process adhere to slip surface plane and keep poor permeability on slip surface.
- Maintained excess pore water pressure enables sliding blocks to move on long distance with high speed.

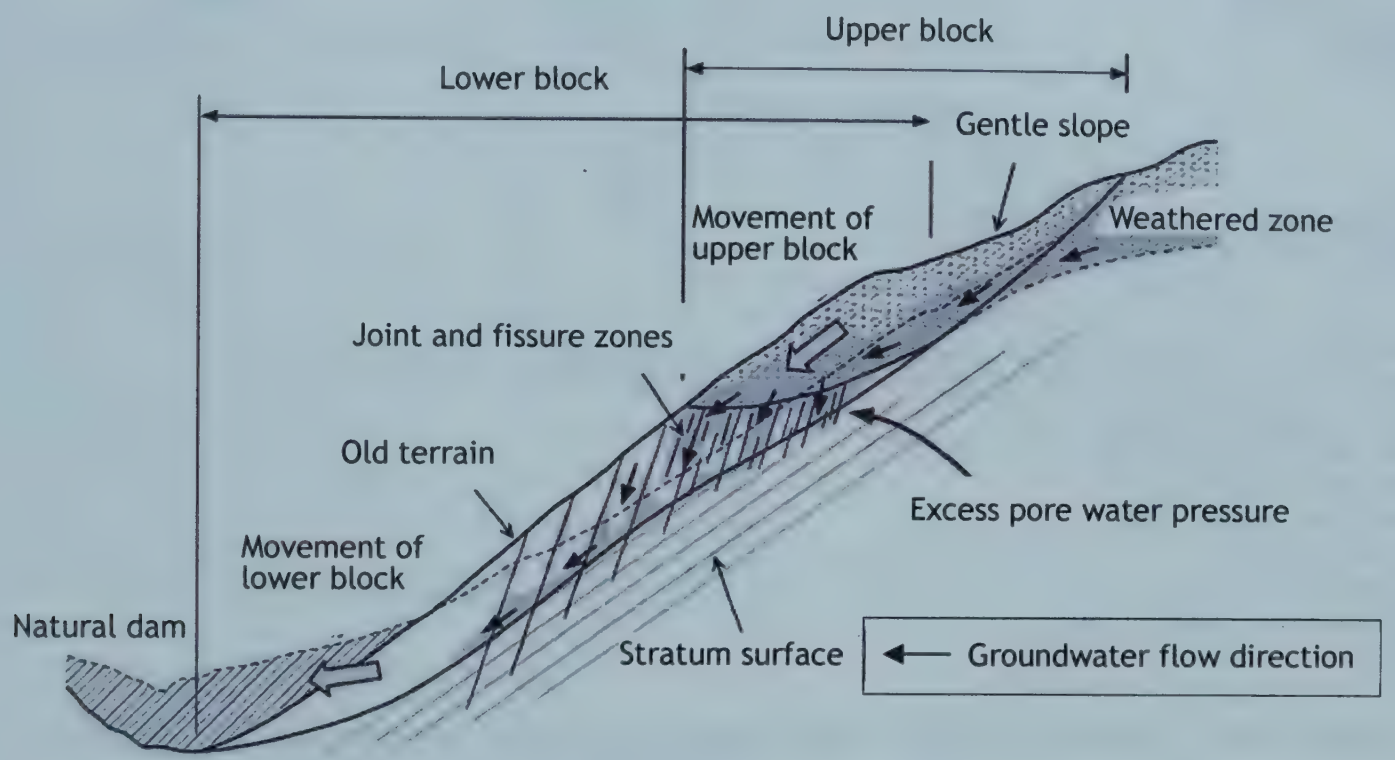


Figure 1-19 Triggering Mechanism of Deep-seated Landslide

Source Kinoshita et al. (2013)

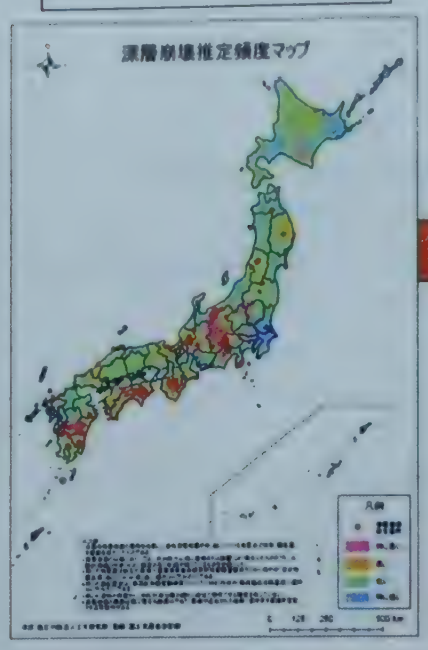
(2) Detecting and Assessing Risk of Deep-seated Landslide

A: Effort by Government of Japan to Indicate Deep-seated Landslide Hazardous Areas

Once it occurs, a deep-seated landslide poses devastating threats to the surrounding areas in a form of not only direct damage caused by fallen debris or large-scale debris flow but disaster risk by landslide dams which may be formed by deposition of fallen debris. In order to mitigate such damage and disaster risk caused by deep-seated landslides, therefore, predicting site and scale of probable deep-seated landslides is required in advance. Among such requirements, prediction of probable sites of deep-seated landslides is practicable to some extent, by filtering the slopes with objective evidences.

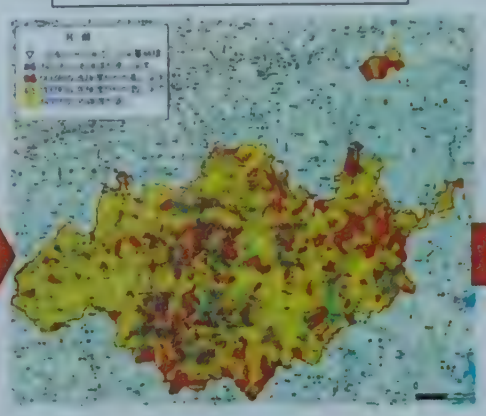
In Japan, in November 2008, Public Works Research Institute (PWRI) of the National Research and Development Agency published "Manual on the method for extraction of torrents prone to a massive landslide occurrence (draft), Public Work Research Institute Data No. 4115". In addition, in August 2010, the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) published "Deep-seated catastrophic landslide frequency map of Japan", which estimates the frequency of deep-seated catastrophic landslides in Japan. In this map, the risk level of deep-seated catastrophic landslide prone areas is evaluated for each torrent referring the above-mentioned PWRI's manual. Furthermore, in September 2012, a torrent level evaluation map was published along with a density map of scars/traces of old deep-seated catastrophic landslides by regions. In March 2016, in order to examine more detailed countermeasures for deep-seated collapse, PWRI summarized the knowledge obtained up to that time and released the study as "The technical note of PWRI No. 4333, Research on methods of extraction technology and risk assessment methods for deep-seated catastrophic landslide prone slopes".

Extraction of hazardous areas



Published by the Ministry of Land, Infrastructure, Transport and Tourism in August 2010 as a map of estimated frequency of deep-seated collapses
Based on actual collapses, the whole country is rated in 4 levels

Extraction of dangerous streams



A manual for stream extraction was published by the Public Works Research Institute in November 2008
Relative evaluation of dangerous areas within a certain area
4 levels for each 1 square km of stream (small streams)

Extraction of slopes where deep seated landslides are likely to occur

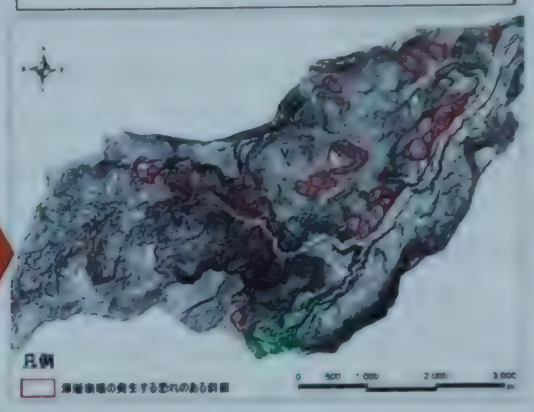


Figure 1-20 Trends in Japan of Approach to Predict Deep-seated Landslides

Source: Kinoshita et al. (2016)

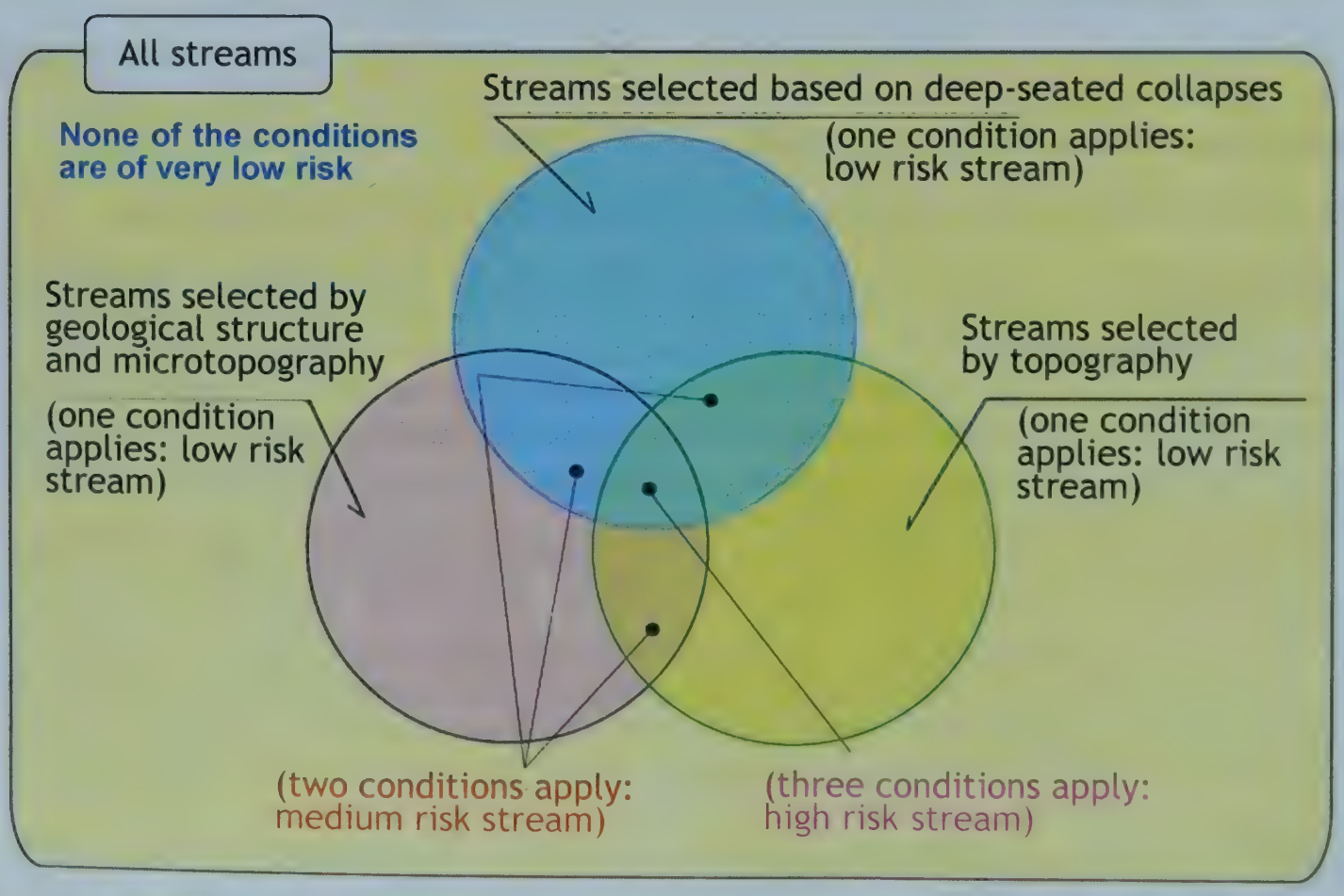


Figure 1-21 Methodology for Identifying Streams at Risk of Deep-seated Landslide

Source: Tamura et al. (2008)

On the other hand, in order to estimate the scale of deep-seated landslides, understanding geological structure of the targeted slopes is required. Thus detailed investigation, including geological field survey, drilling investigation, and geophysical survey are conducted to estimate the spatial distribution of slip surface and, based on it, the amount of displaced mass.

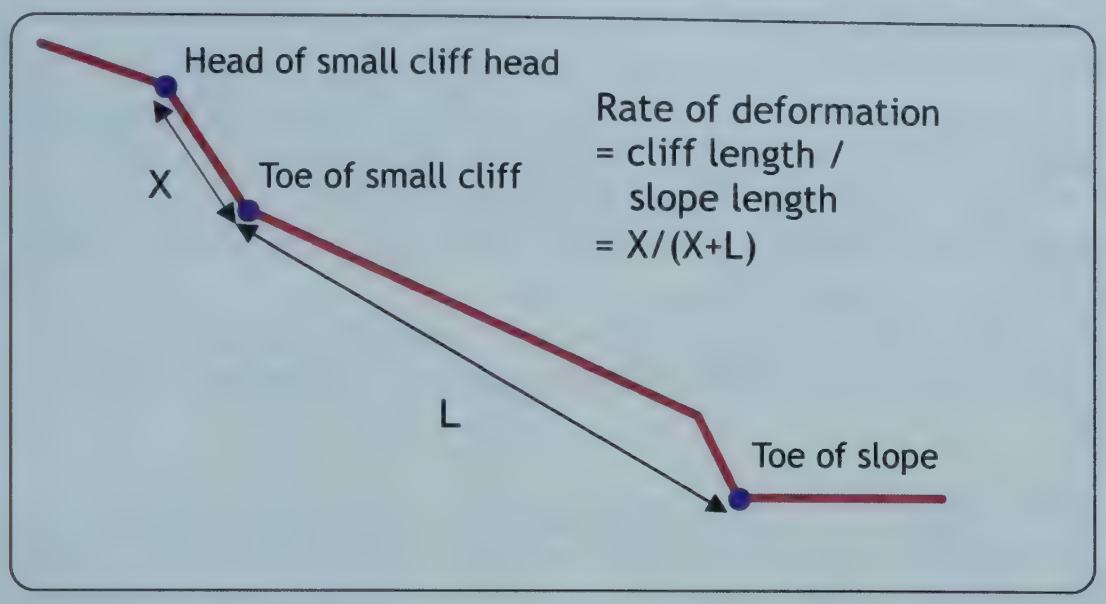


Figure 1-24 Calculation of Deformation Rate

Source: Kinoshita et al. (2016)

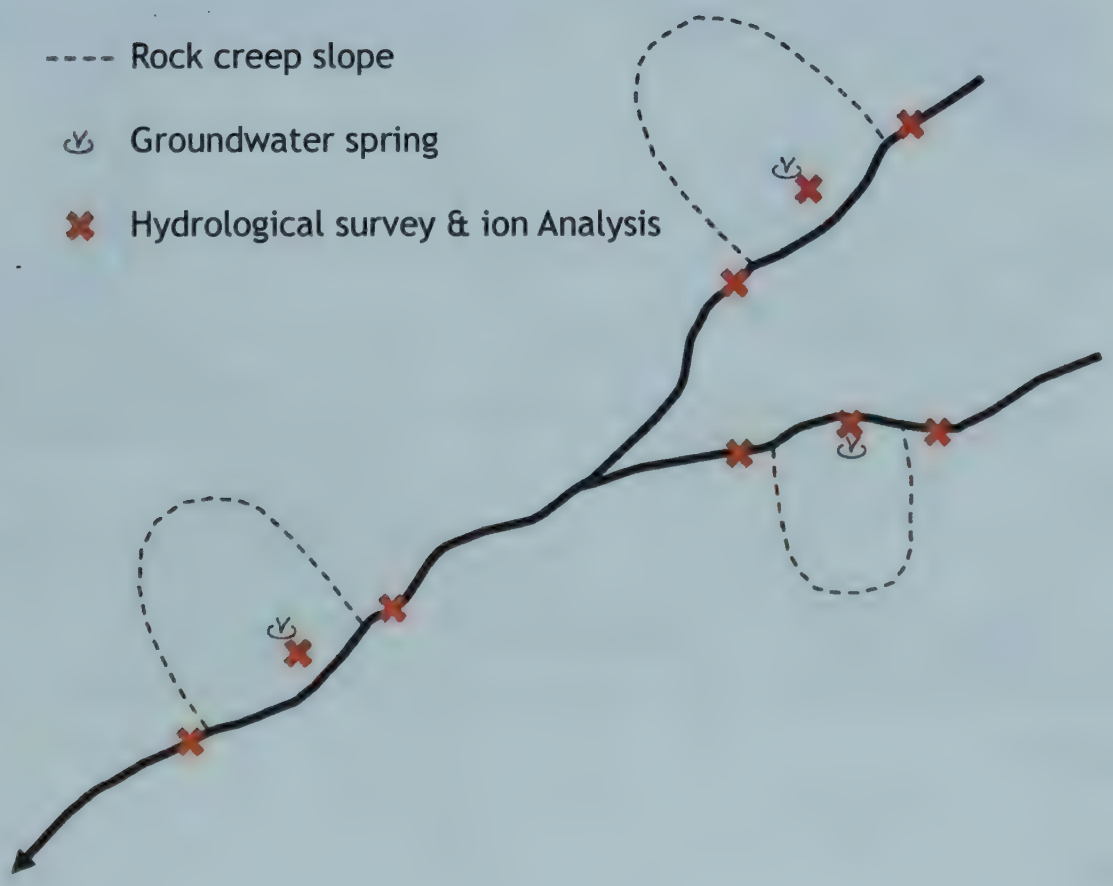


Figure 1-25 Plan for Hydrological Survey and Ion Analysis

Source: Kinoshita et al. (2016)

B: Estimation of Depth of Deep-seated Landslides with Airborne Electromagnetic Survey

For the slopes with high priority, detailed investigation including drilling investigation and geophysical survey should be conducted. Among the geophysical survey, tomographic analysis of resistivity survey or airborne electromagnetic survey are effective to obtain resistivity image of the ground. Then, with geological field survey and drilling investigation, the resistivity image is interpreted to understand subsurface condition.

Seto et al. (2016) proposed a method of estimating the depth of slip surface of possible deep-seated catastrophic landslides from resistivity images obtained through airborne

electromagnetic survey. Seto et al. focused on the fact that the resistivity obtained by airborne electromagnetic survey reflects the clayey contents and volume moisture content (= porosity × degree of saturation).

They considered that the depth at which a resistivity curve clearly shows its change indicates the depth at which geological and groundwater conditions change; the change of resistivity means hydrogeological discontinuities. They proposed a resistivity image processing technique called "edge detection", which uses a differential filter to determine the depth at which the resistivity changes clearly. In the analysis at Mt. Kurikomayama landslide area shown in Figure 1-26, the depth of 34 meters estimated by the secondary differential filter corresponds to the depth of 28 meters where the geological boundary between andesite and altered tuff breccia lies. And the depth of 54 meters is similar to the depth of 51meters where the geological boundary of altered tuff breccia and tuff breccia lies.

Seto et al. claims that the "edge detection" can be applied to extracting slopes with risk of deep-seated landslides with lateral analysis and to estimating volume of landslide mass with vertical analysis.

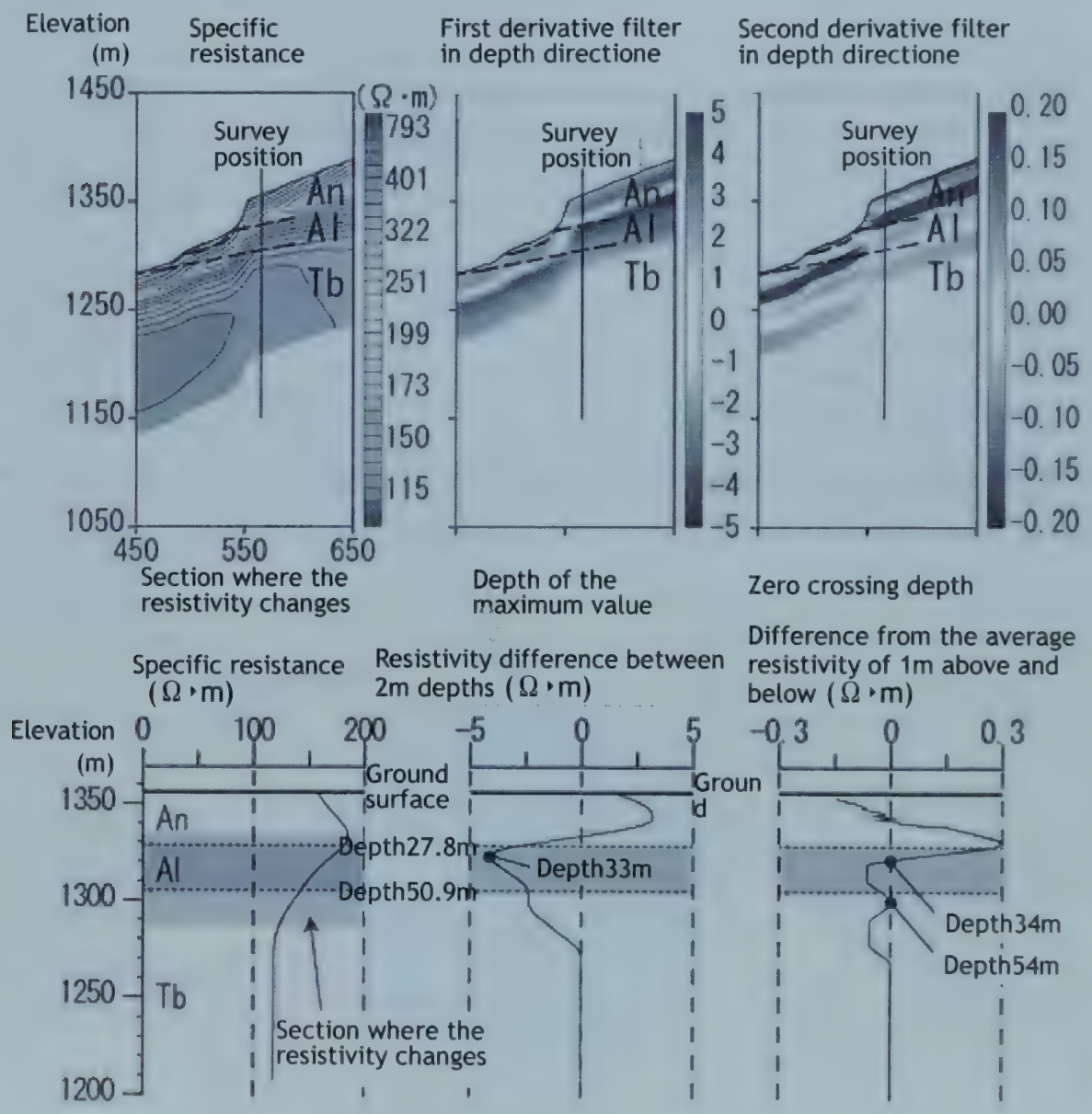


Figure 1-26 Application of Resistivity Edge Extraction Method at Kurikoma Mountain Collapse Site

Source: Seto et al. (2016)

In addition, Takahara et al. (2016) compared the results of airborne electromagnetic surveys conducted in an area with a high risk of deep-seated catastrophic landslide in Nagatono, Totsugawa village, Nara Prefecture, with the geological data obtained by seismic explorations and drilling surveys. Based on the comparison, Takahara et al. reported that underground structure interpreted by resistivity survey well reflects band of weathered zones and distribution of groundwater.

The left column of Figure 1-27 shows the electrical resistivity profiles. The right column shows the results of calculation by secondary differential filtering; in right column, the figures clearly indicate the depth where electric resistivity changes.

The resistivity image on the left is divided into two layers; a low resistivity in the shallow layer and a high resistivity in the deep. It is clearly understood that the line connecting the inflection points of the resistivity contour running close to the boundary of the upper and lower layers almost corresponds to the line of groundwater level.

Similarly, in the profiles based on the calculation by secondary differential filtering on the right, for the depth where electric resistivity changes clearly by the secondary differential filtering, the depths where resistivity change to negative side well corresponds to the line of groundwater level.

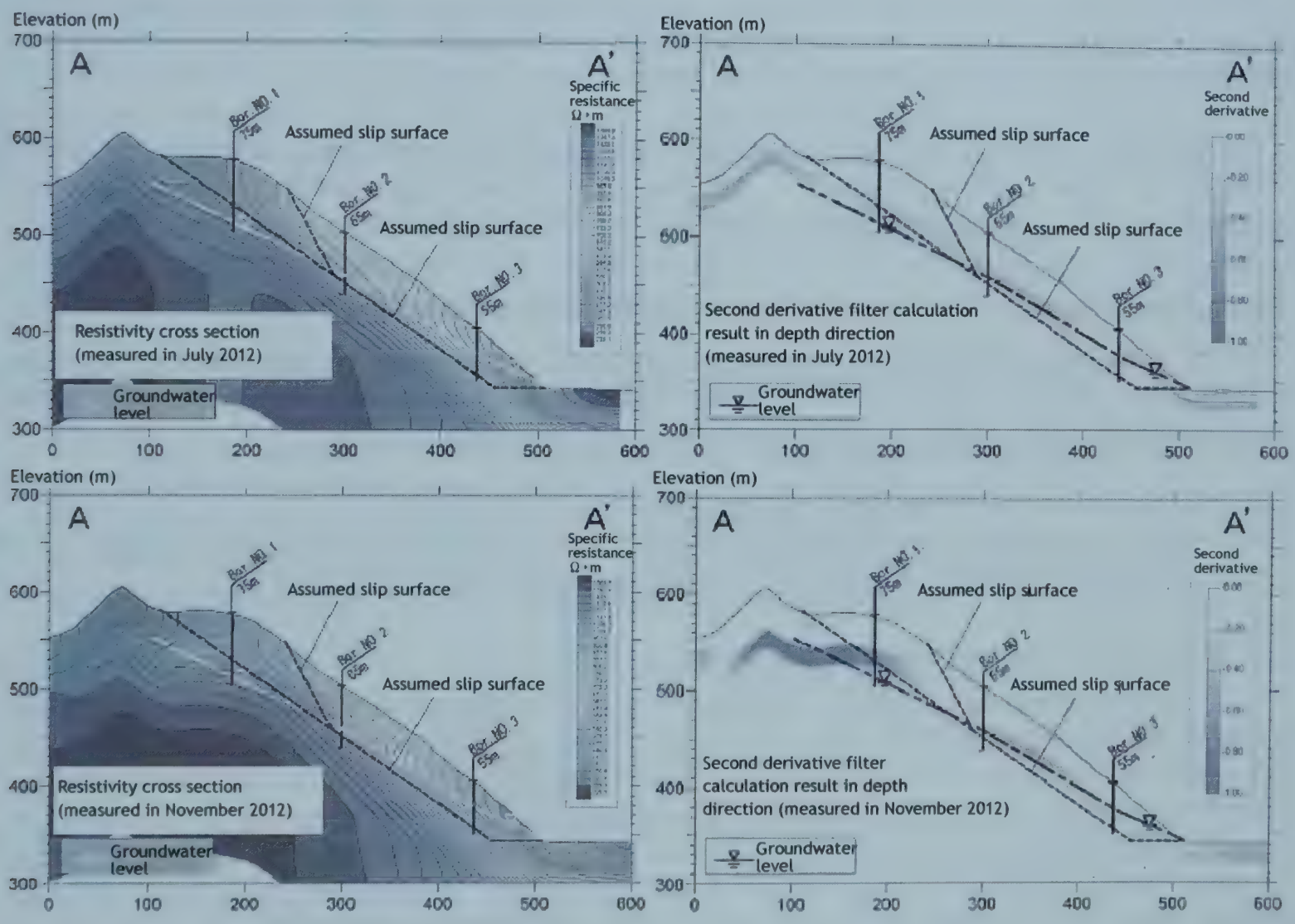


Figure 1-27 Resistivity Profile and Resistivity Rapid Change Depth Representation

Source: Takahara et al. (2016)

1.3.2 Countermeasures against Deep-seated Landslide and Gravity Deformation

In Japan, although it is well understood that gravity deformation is a phenomena that is extremely related to the fields where deep-seated landslides often occur, physical countermeasures against gravity deformation is yet firmly established.

Before establishing practical countermeasures against gravity deformation, there are various problems to be solved. Firstly deep-seated landslides, which are induced by heavy rainfalls or earthquakes, generally occur simultaneously. Secondary, among the slopes where gravity deformation are in progress, survey technologies for prioritizing slopes for countermeasure implementation are not established. Thirdly, the steep inclination of slip surfaces limits the effect of conventional countermeasures against slow-moving landslides.

On the other hand, due to the climate change by so-called global warming, in recent years, the risk of deep-seated landslides are considered increasing. To protect human lives and infrastructure, passive countermeasures against deep-seated landslides are implemented in advance, placed just in front of the objectives to protect.

In this section, the countermeasures against deep-seated landslides currently implemented in Japan are explained. At the same time, the development of potential countermeasures in future against gravity deformation are overviewed.

(1) Basic Principles of Countermeasures against Deep-seated Landslides

In 2014, the National Institute for Land and Infrastructure Management organized Technical Note No. 807 as a future reference for countermeasures against deep-seated landslides. The basic technical items of the countermeasures are as follows:

- ✓ "Structural countermeasures by implementing facilities",
- ✓ "Soft countermeasures depending on provision of information on urgent danger",
- ✓ "Soft countermeasures provided as mapped information on risks of land"

The text at the beginning of this Technical Note is worth to read and thus quoted below:

> Deep-seated catastrophic landslides are less frequent than ordinary landslide disasters. Deep-seated landslides are very large and may cause enormous damage, however. On the other hand, conventional erosion control facilities may not have sufficient bearing capacity or resistance against deep-seated catastrophic landslides or landslide dams caused by deep-seated catastrophic landslides. In addition, the information on the risk of land provided by the basic investigations stipulated in the Sediment-related Disaster Prevention Act may not sufficiently cover such disasters caused by deep-seated catastrophic landslides. In other words, conventional countermeasures against ordinary landslide disasters are likely to be insufficient to prevent such damage caused by deep-seated catastrophic landslides.
>
> In order to reduce damage caused by deep-seated catastrophic landslides, therefore, it is important to take all measures including "Structural countermeasures by implementing facilities", "Soft countermeasures depending on provision of information on urgent danger", and "Soft countermeasures provided as mapped information on risk of land".

Based on the above, for formulating basic policy for countermeasures, points of concern are indicated as follows:

> In the areas where risk of deep-seated catastrophic landslides is considered high, the disaster scenario for such area should expect deep-seated catastrophic landslides to occur. Based on the scenario, basic policy on countermeasures should be established, in which structural and nonstructural measures are combined effectively to cope with the deep-seated catastrophic landslides expected in the area.

In this section, the structural measures referred in Technical Note No. 807 are explained. In Technical Note No. 807, structural measures are described separately depending

on each disaster scenario. Regarding deep-seated catastrophic landslides, the following three basic disaster types are supposed.

A: Landslide / natural dam type
B: Debris flow type
C: Direct hit by collapsed rock mass type

A: Landslide / Natural Dam Type

A landslide dam, or natural dam, involves a large amount of debris and water flowing down and being deposited downstream. The external force may become extremely large at the toe of a deep-seated landslide. A range to be affected by a landslide dam is often larger than those by debris flow or by direct hit by collapsed rock mass.

On the other hand, debris flow caused by collapse of a landslide dam includes less debris. Thus facilities to control debris flow or flood caused by collapse of a landslide dam should properly treat mud water while trapping debris. At immediately downstream of a landslide dam, debris flow or flood is supposed to show a particular pattern with a large peak discharge in short time period.

In addition, landslide dam has two types; instant destruction type or long lasting type. For the latter one, in addition to preventive measures prepared beforehand, emergency measures after formation of a landslide dam should be implemented.

Types of preventive structural measures against landslide dam are listed as follows:

1) Facilities to control / prevent occurrence of deep-seated landslides
2) Facilities to reduce the height of landslide dams to be formed
3) Facilities to prevent sudden collapse of landslide dams
4) Facilities to control debris flow or mud flow from collapsed landslide dams
5) Facilities to support prompt implementation of emergency countermeasures

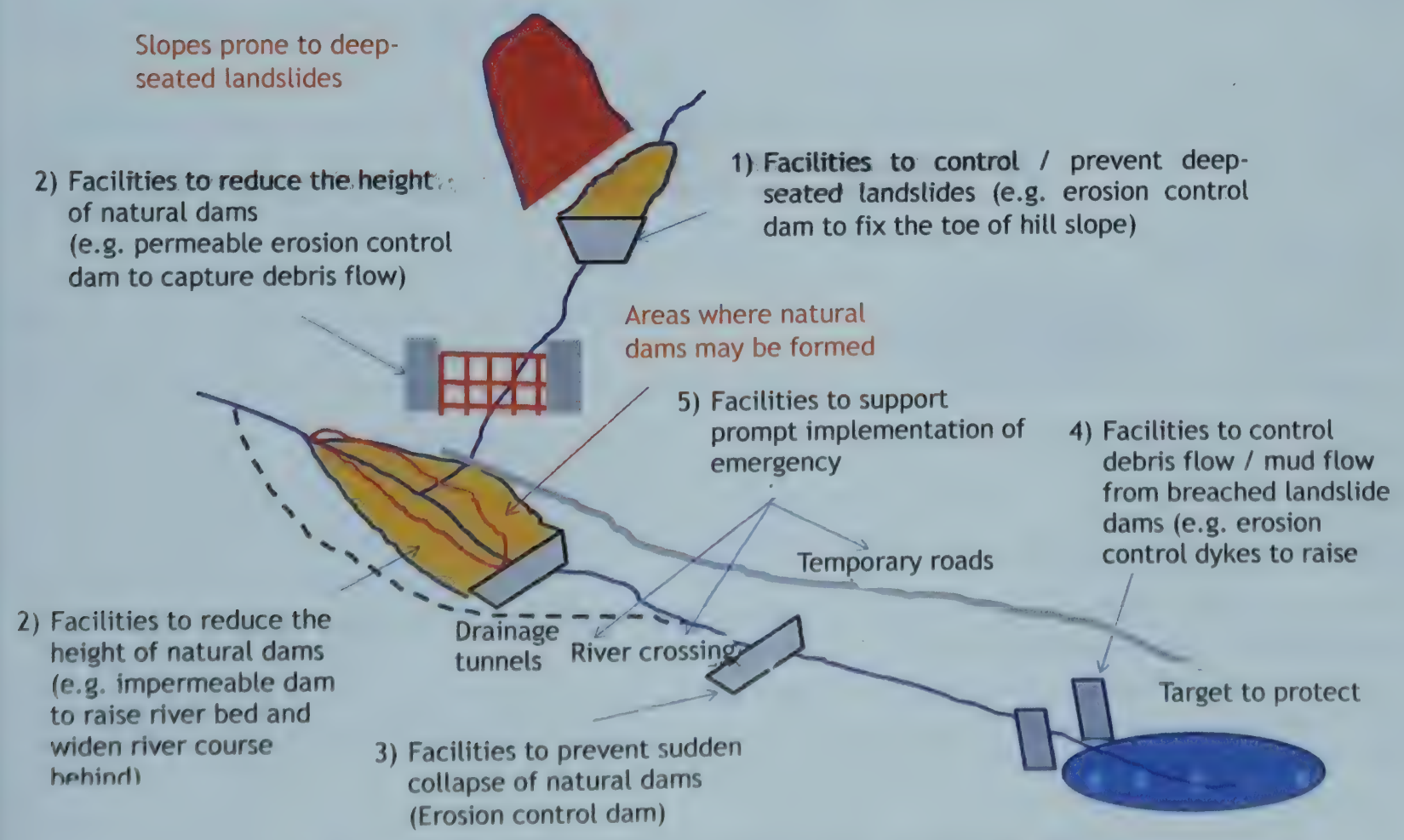


Figure 1-28 Combined Countermeasures against Natural Dam

Source: National Research Institute data (2014)

B: Debris Flow Type

Debris flow also involves a large amount of debris and water flowing down and being deposited downstream. The external force may also become extremely large at the toe of a deep-seated landslide. A range to be affected is often larger than those by direct hit by collapsed rock mass but smaller than those by a landslide dam.

The debris flow directly caused by deep-seated landslides includes more debris. Thus debris immediately starts depositing when inclination of a stream becomes gentle and the external force in upstream area remains extremely large. Thus construction of facilities in upstream area is quite difficult, therefore. Thus countermeasures should utilize the distance between risky slopes of deep-seated landslides and targets to protect.

Types of preventive structural measures against debris flow type are listed below:

1) Facilities to control / prevent occurrence of deep-seated landslides
2) Facilities to capture debris flow caused by deep-seated landslides
3) Facilities to reduce scale of debris flow

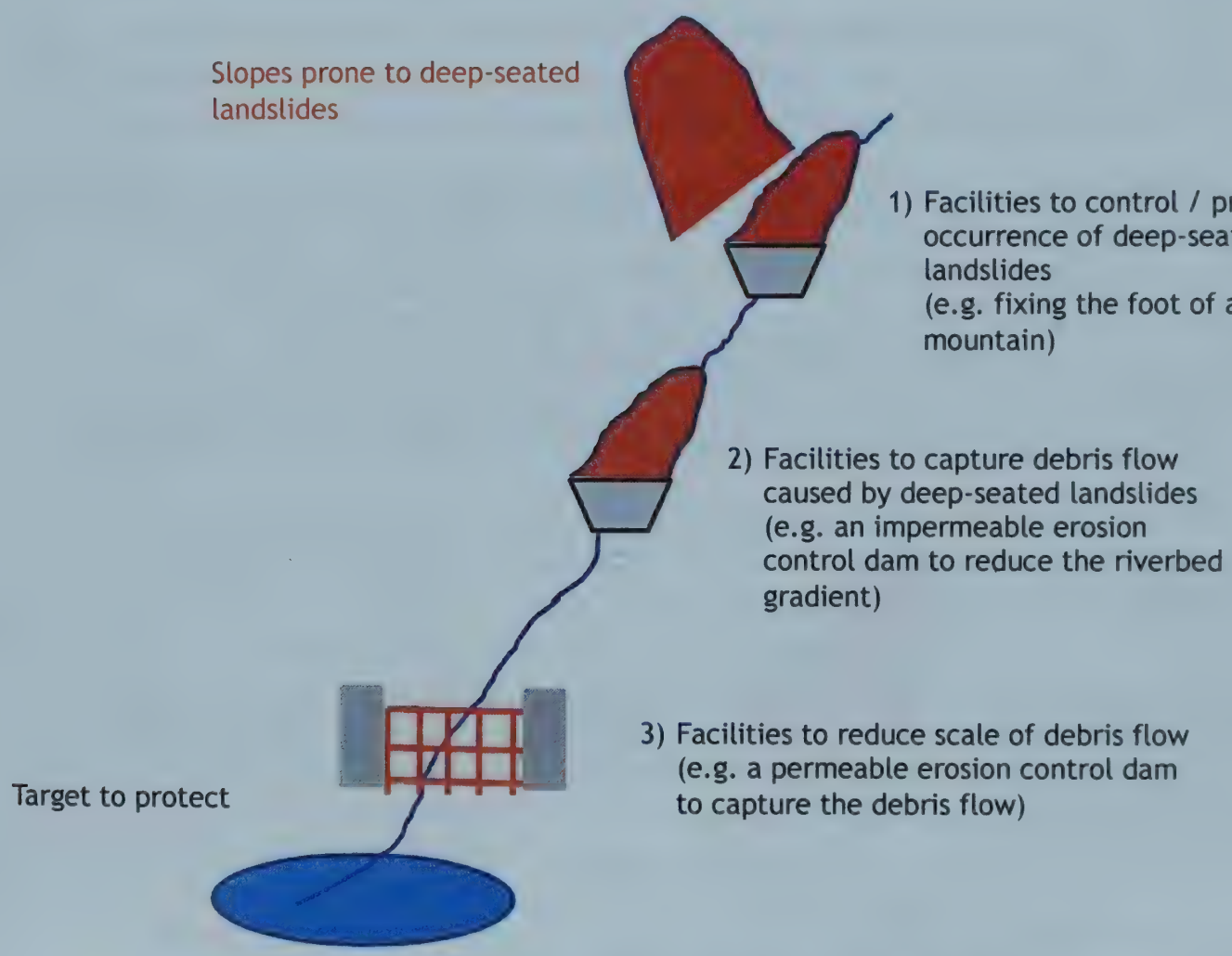


Figure 1-29 Countermeasures against Debris Flow Type

Source: National Research Institute data (2014)

C : Direct Hit by Collapsed Rock Mass Type

A range to be affected by direct hit by collapsed rock mass is often smaller than those by landslide dam and debris flow.

The countermeasures against direct hit should be placed within the limited areas between risky slopes of deep-seated landslides and targets to protect. Thus countermeasures should be placed on each risky slope of deep-seated landslides.

Types of preventive structural measures against debris flow type are listed below:

1) Facilities to control / prevent occurrence of deep-seated landslides
2) Facilities to reduce damage by deep-seated landslides

Except measured to control development of gravity deformation, technologies for the countermeasures to control / prevent deep-seated landslides are not well developed.

Deep-seated landslides caused by development of gravity deformation is triggered by sudden collapse of the toe of or the entire part of a gravity deformation block. Thus, before such triggers occur, implementing countermeasures to obstruct /control development of gravity deformation is considered effective.

(2) Structural Measures Expected to Control Gravity Deformation

Different from the movement of ordinary landslides, in which slide occurs in a particular sheared / slip surface within a thin layer of landslide clay, the movement of gravity deformation is a creep deformation of loosened zones of bed rock, in which loosened zones with certain thickness deforms entirely. Slip surface of ordinary landslide with landslide clay is governed by shear characteristics of cohesive soil. On the contrary, gravity deformation is governed by friction of fissures within loosened zone of rock mass. Thus internal friction angle of loosened zones are very large as 30 to 40 degrees. As a result, the inclination of slip surface of gravity deformation is steep as well. Subsidence of the ground is remarkably noticed as a result of deformation.

Ground anchor works and pile works are common prevention works against ordinary landslides with landslide clay in Japan but are less effective for controlling gravity deformation. The preventing effects by both works are reduced when the inclination of slip surface is steep. On the other hand, reinforced earthworks are applied against failure or deformation on steep slip surface. Rock bolts and soil nailing are among reinforced earthworks.

In Japan, as countermeasures against slides / failures on steep slopes or stabilizing works for the same, concrete crib works with rock bolts and rock bolts with bolt head connection are often applied. In the Japanese design requirements, however, the maximum applicable depth of slip surfaces for the design of rock bolts is 3 meters and the maximum length of rock bolts are 6 meters. On the other hand, soil nailing in India, which is similar to rock bolts, can be applied to the slope failures with steep slip surfaces much deeper than 3 meters. So the preventive works by installing rigid steel bars to steep slopes may have a chance to control gravity deformation with steep slip surface, as long as slip surface is not so deep.

Depending on the geological conditions, the depth of loosened zones of gravity deformation should vary and the necessary length of soil nailing to control gravity deformation should vary as well. The maximum applicable length of soil nailing for such purpose should be well studied, however.

On the other hand, unlike ordinary landslide with landslide clay, a gravity deformation block is mostly composed of bed rock without particular thin slip surfaces. Up to a certain depth, loosened rock mass can be effectively tightened by homogenizing effects of rock slope provided by installing rigid steel bars with head connection. Then deformation by loosening can be reduced.

Inclined Pile Group Method proposed by Oda et al. (2011) was developed to stabilize the toe of an ordinary landslide block and is featured by bearing plates on the heads of micro piles. With its purpose of development, Inclined Pile Group Method has a potential to control gravity deformation by homogenizing effects.

In general, even after countermeasure works are implemented, the toe of an ordinary landslide is often unstable. A toe of a landslide is often affected by driving force from behind and deformed into various forms, such as multi-layered shear zones caused by passive failure and loosened rock mass with toppling structure. Conventional countermeasures such as pile works or ground anchor works are designed to control a homogenized sliding body by applying large preventive force at a particular zone. Thus stabilization over an entirely unstable slope is not suitable for such works. In addition, at a toe of a landslide, depths of unstable parts are deeper than ordinary slope. Inclined Pile Group Method aims to stabilize entirely unstable slope with deeper slip surface, which is similar to a gravity deformation slope.

A series of tests were carried out by Oda et al. to prove effects of Inclined Pile Group Method:

- ✓ Airborne electromagnetic survey to detect actual condition of loosened zone
- ✓ Identification of loosened zone with ground penetrating radar.
- ✓ Test construction to verify recommendation for construction
- ✓ With strain gauges attached directly on the micro piles installed in the test construction, pull-out load tests to identify the points where pull-out resistance is expressed.
- ✓ With a number of soil pressure gauges installed in the test construction, loading tests to verify the effect by shape of bearing plate on load distribution.
- ✓ Model experiment considering size effect to examine appropriate material and pile spacing, effect of bearing plates, and the effect of reinforcing force by integration of piles and ground.
- ✓ Finite Element Method (FEM) to analyze consistency between the results of model experiments and field observations as well as supporting mechanism of micro piles in actual cross sections.

Although Inclined Pile Group Method is still in the research stage, potential of the method to control gravity deformation is well proven by its purpose of development as well as effectiveness of design concept.

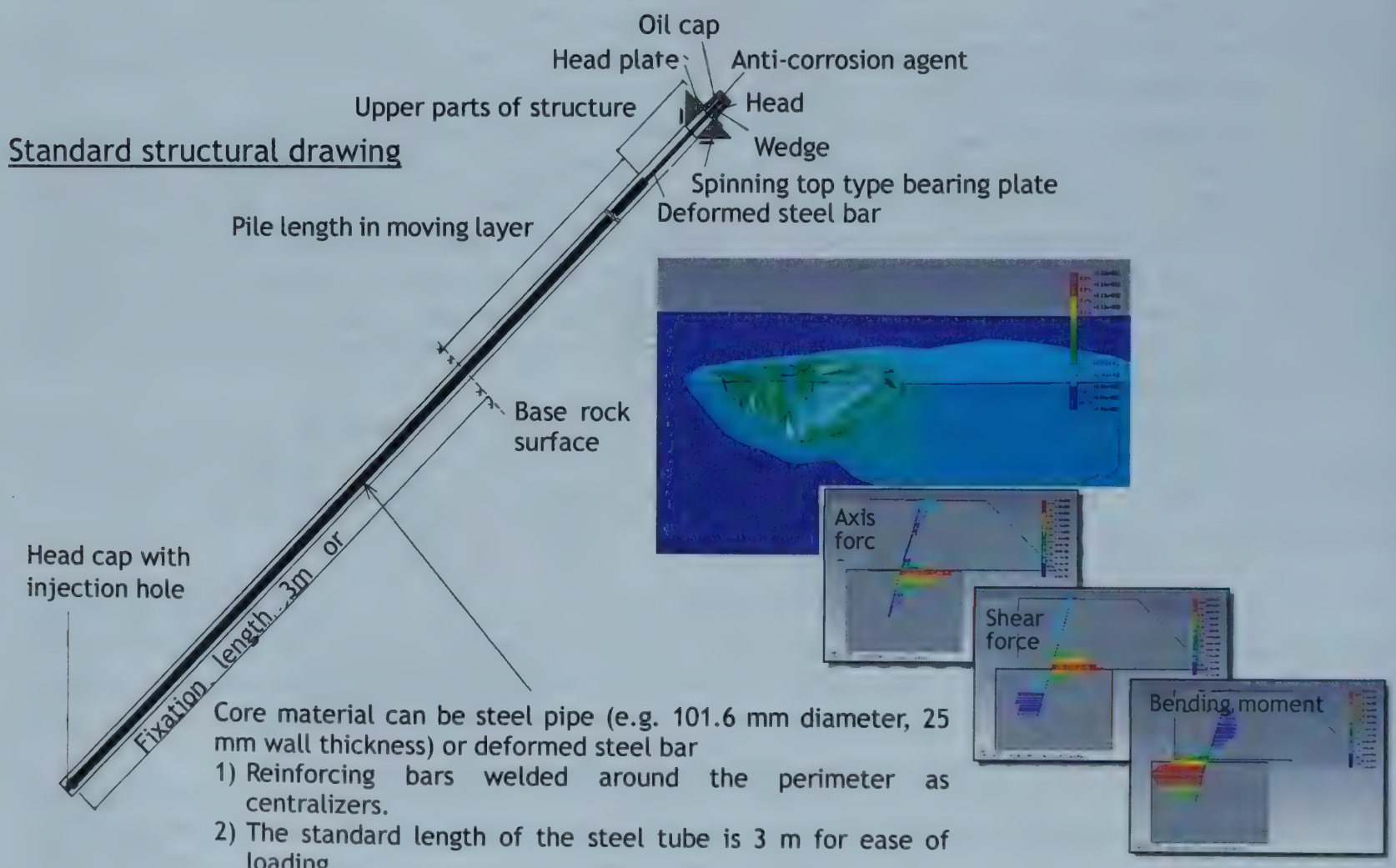


Figure 1-30 Structural Drawing and FEM analysis of Inclined Group Piling Method

Source: Oda et al. (2011)

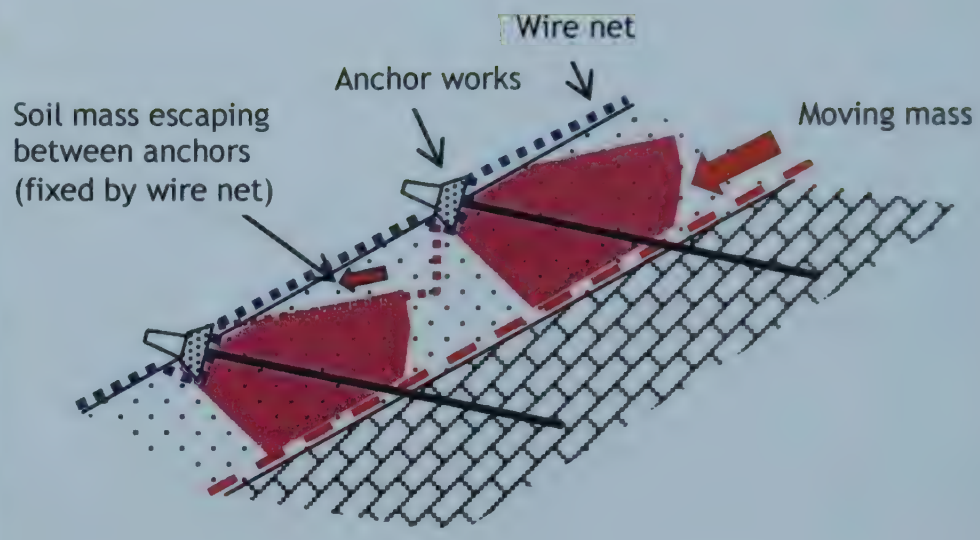
Adopting the theory of Inclined Pile Group Method, in which bearing plates are equipped on heads of micro piles, Japanese suppliers developed a series of materials for the method. Such materials are used in Japan and thus designed to stabilize the slopes with slip surface of 3 meters deep or shallower. Considering the potential of the method, however, with future application outside Japan, the method is expected to be applied to gravity deformation slopes with medium or deep loosened zones.

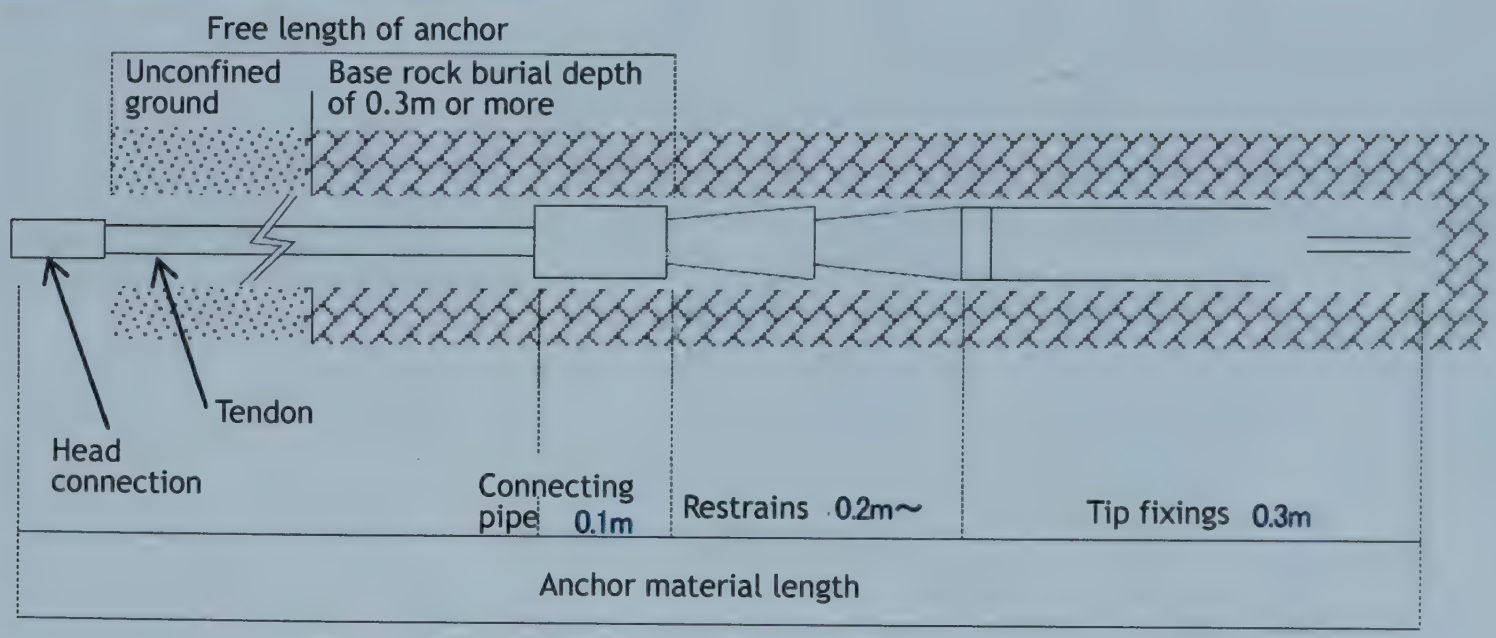
Figure 1-31 Steel Pipe Pile Method for Preventing Surface Collapse Miraforce I

Source: Tokyo Rope Inc.

Anchor net method

Published patent bulletin 2010-174598





Free length of anchor

Structural Drawing of Anchor for Deep Layers

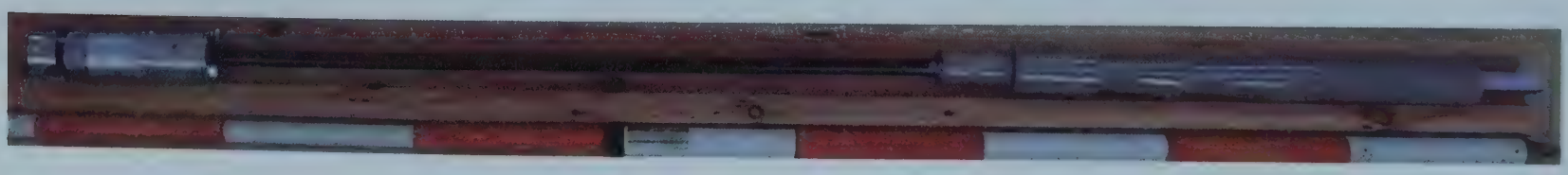


Figure 1-32 Stainless Steel Wire Ring Anchor Net for Protection against Rock Slope Failure

Source: Daika Corporation

(3) Control Measures Expected to Control Gravity Deformation

As control measures against gravity deformation, the following measures are expected:

- Groundwater drainage works, such as horizontal drainage boring
- Soil / rock removal from the upper slope of gravity deformation blocks
- Counter embankment to the lower slope of gravity deformation blocks

Conditions for slope stability analysis and principle of design are summarized in Chapter 2.6.6 of JICA Slope Guideline.

CHAPTER 2 Spotting Gravity Deformation through Map Reading

2.1 Risk Assessment through Map Reading

2.1.1 Role of Map Reading in Risk Assessment

Importance of risk assessment and detection of hazards is increasing worldwide. With the development of spatial information technology, topographic information has become highly accurate and widely used in various purposes. At the same time, development in GIS technology has made it possible to process subsurface information and social conditions objectively and quantitatively on a grid-by-grid basis. As a result, the information on surrounding hazards or risks is easily recognized as spatial polygons with coordinates.

On the other hand, topographic contour map is a general and widely used method to express shape of terrain with shape and density of contour lines. In the recent days in which topographic data is processed in grid units, however, topographic contour maps are tend to be recognized as a background layer for GIS products. And the technique to "extract geomorphological information from the shape and density of contour lines of topographic maps" seems to be almost forgotten.

The authors have a serious concern about the above-mentioned trend. In the topographic map with contour lines, dynamic geomorphological history of a certain place from the past to the present is expressed by contour lines. Thus interpretation through map reading is not only to spot the sites where landslides are anticipated, but to understand background behind such landslides, impacts by such landslides to its surroundings, and future events such landslides may induce.

The modern spatial information technology relies on information attributed to grids. No matter how much information attributed to each grid, it is not easy to build a dataset while maintaining time-series relationship among related grids. On the other hand, a well-trained engineer can interpret geomorphological history of terrain from shape and densities of contour lines.

At the beginning of this chapter, the authors would stress the importance to understand the principles of terrain formation, specifically, to recognize geomorphological events with scenarios on the time line from the past to future.

On the other hand, utilizing the cutting edge technologies can be used to verify the results of aerial photo or map reading. The risky areas of landslides and gravity deformation extracted thorugh map readings utilizeing contour maps produced from DEMs include traces of all geomorphological events from the past to the present. Some of the traces may already be stabilized after active movements and thus potential of such traces to cause landslide disasters may not be so high. Thus risky areas interpreted through aerial photo or map reading

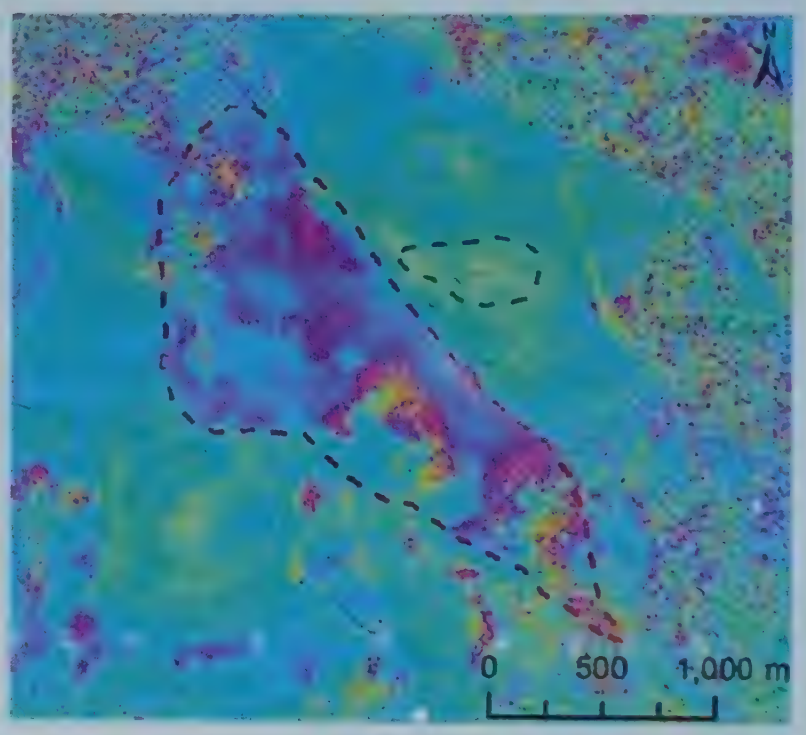


Figure 2-1 Landslide Activity Evaluated by SAR Interferometry Analysis

Source: JICA Project Team

should be scrutinized by secondary filtering, in the aim of extracting the risky areas that keep potential to cause landslide disasters. So far, such secondary filtering has been made for each risky area through on site evaluation in field survey. In recent years, however, Synthetic Aperture Radar (SAR) technology is utilized for such purpose. With periodic observation, SAR can provide disaster potential of slopes or torrents with scientific evaluation of displacements observed among different timings.

Even it should be filtered on site evaluation or remote sensing SAR technology, map reading with contour map is basic and reasonable technology to detect risky landslide and gravity deformation areas in hill road development projects as well as operation and maintenance of hill roads. With free DEM data explained in Chapter 1.1.8 of JICA Slope Guideline, topographic contour maps of a certain area can be produced and used for the risk assessment through map reading explained in this chapter.

2.1.2 Ridges and Valleys Drawn by Contour Lines

In general, a topographic maps expresses undulations of terrain in a wide area with contour lines. Contour lines connect the places on the same elevation continuously. The interval in elevation between neighboring contour lines depends on the scale of a topographic map. Figure 2-2 shows a typical topographic map. In addition to the contour lines showing the landform, Figure 2-2 shows symbols of cliffs, elevations of major peaks, rivers with constant water flow, revetment structures, bridges, roads, power transmission lines, land use, vegetation, and other geospatial information, whereas surface of water is given blue color.

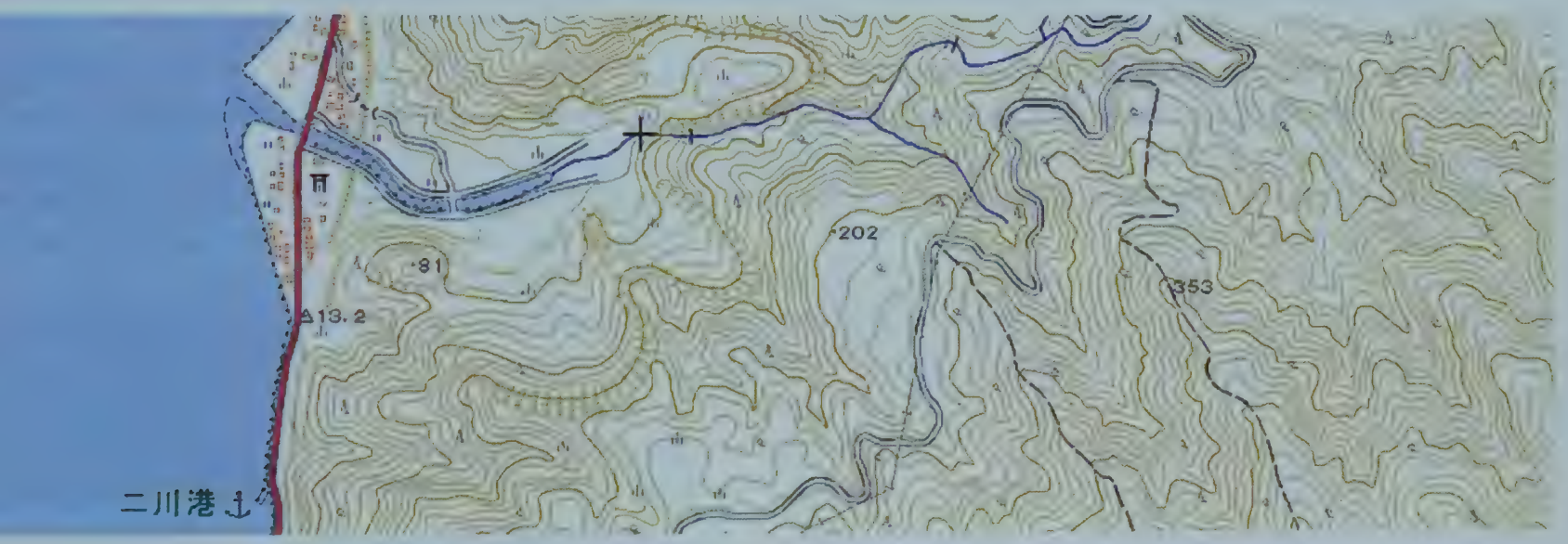


Figure 2-2 Topographic Contour Map

Source: Geospatial Information Authority of Japan

Figure 2-3 shows a 3D bird's-eye view of ridges and valleys. Figure 2-3 also shows waterfall lines which connects the maximum slope directions, knick lines, and concave knick lines. Ridge is a sequence of higher places of a hill between two valleys, while a valley is a sequence of lower places of hill slopes between two ridges.

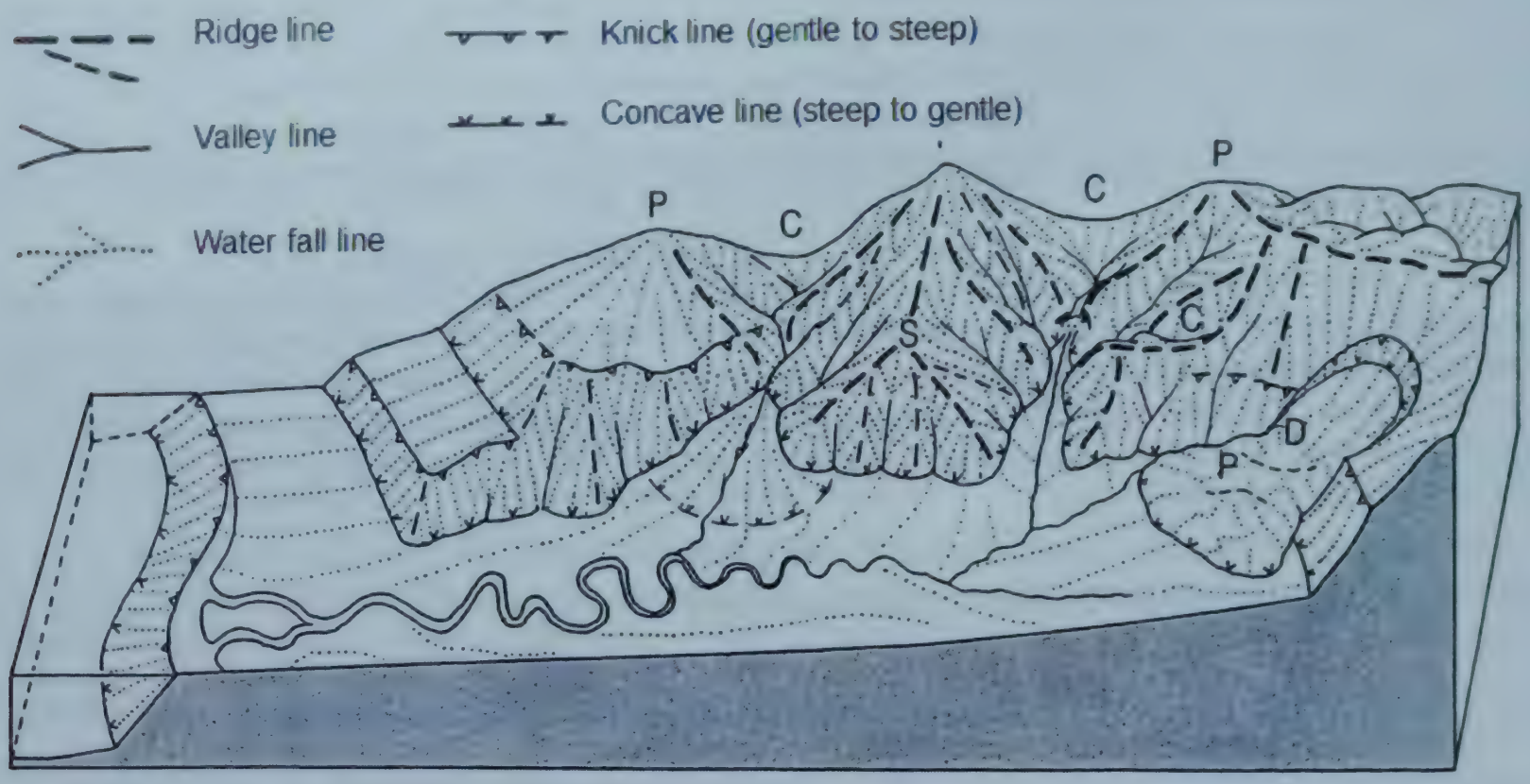


Figure 2-3 Illustration of Ridges and Valleys

Source: Suzuki, T (1997)

How to extract ridges and valleys on topographic maps is as follows (see Figure 2-4):

1) To extract peaks in the target area,
2) To connect tips of convex shaped contour lines seen from the peaks to draw ridge lines, and
3) To connect bottoms of concave shaped contour lines to trace flow paths/valley lines/water system.

In other words, connecting the paths of lower elevations makes flow paths/water system, and connecting paths of higher elevation makes ridge lines.

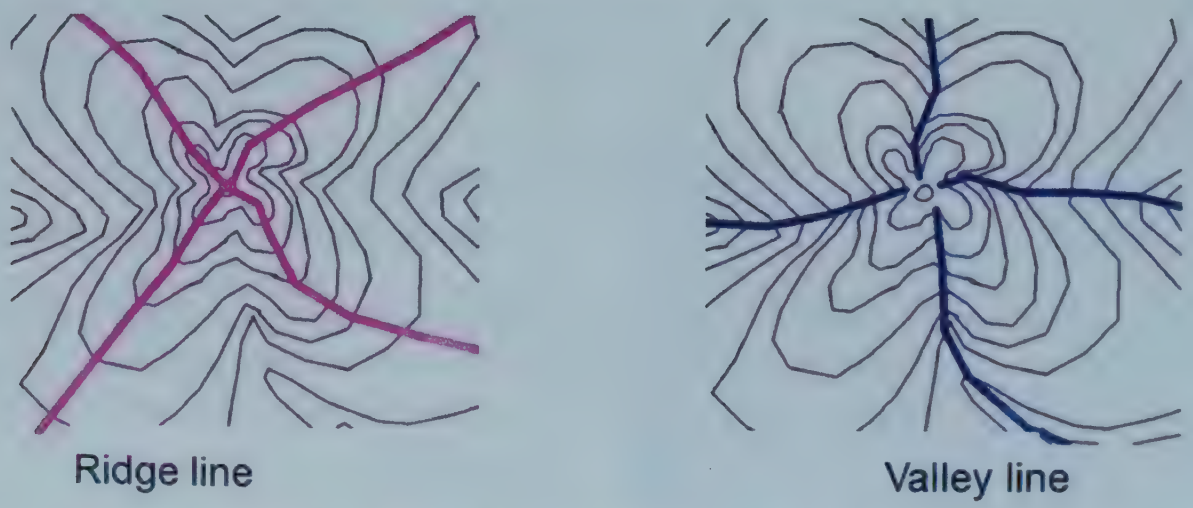


Figure 2-4 Method to Extract Ridge and Valley Lines on Topographic Contour Maps

Source: JICA Project Team

2.1.3 Steep Slopes Expressed by Contour Lines

As mentioned above, contour lines connect the places on the same elevation continuously. As shown in Figure 2-5, therefore, density of contour lines shows steepness of slopes. Where a slope is steep, contour lines run densely. And where a slope is gentle, contour lines run less densely.

The term "steep slope" is defined in Japan as slopes with a gradient of 30 degrees or steeper with a vertical height of 5 meters or higher.

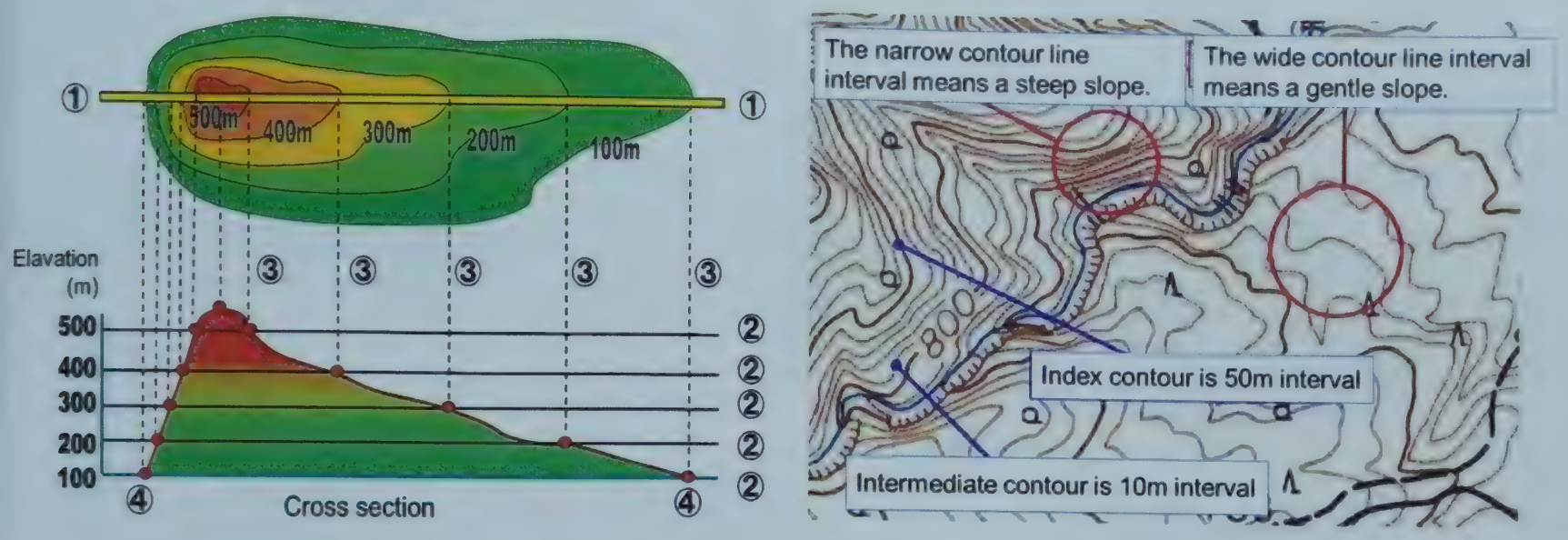


Figure 2-5 Steep slopes displayed by contour lines

Source: JICA Project Team

2.1.4 Landslides Extracted by Contour Lines

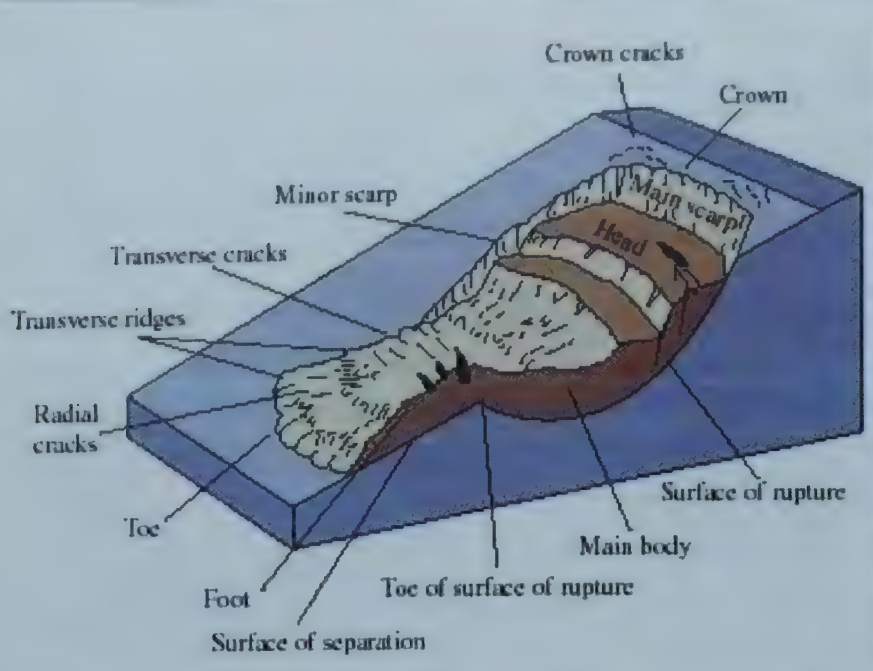
As shown in Figure 2-6 (a), since the main sliding / deforming mass lies on the slope where it is originated, landslides and gravity deformation show distinctive difference in the shapes of contour lines from those of deep-seated landslides/failures, shallow failures, and traces of flow pass of /eroded areas by debris flow, which the sliding mass was fallen away and does not remain on the slope originated.

Both landslides and gravity deformation have clear head/main scarps at its head as well as steep slopes at its toe caused by bulging due to compressive deformation. On the other hand, a landslide often forms a gentle slopes between head scarps and steep slopes at the toe. Gravity deformation, on the contrary, rarely forms such distinctive gentle slopes between head scarps and the toe of deformation. Such difference between landslides and gravity deformation is seen in elevation contour lines as well.

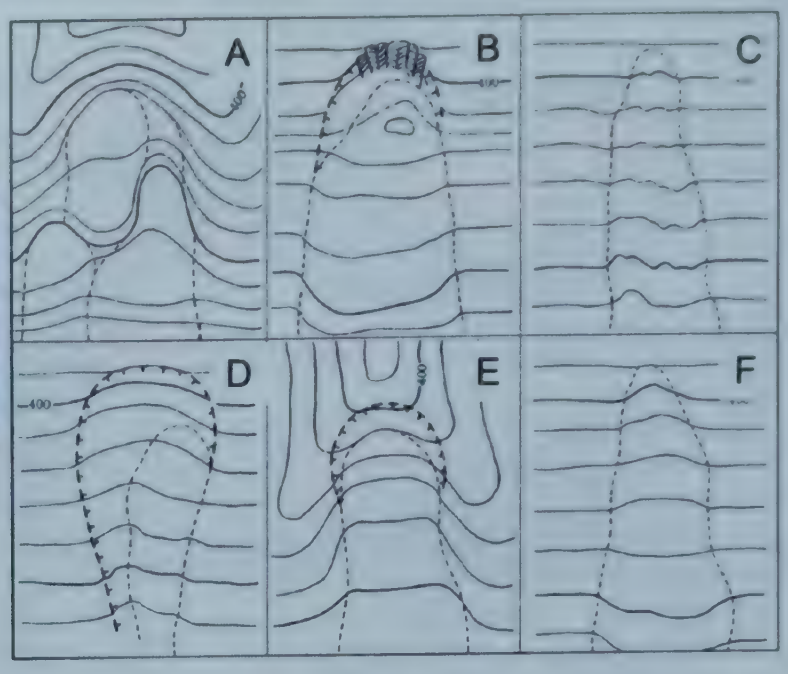
In the areas of landslides, densely running contour lines are seen at head scarps. Then the contour lines similar or less densely to the surrounding slopes are seen between head scarps and toe of slides. Then densely running contour lines are seen at the toe. In the areas of gravity deformation, densely running contour lines are seen at head scarps as well as toe of the slope. The slopes between head scarps and the toe are also steep with a gradient of 30 degrees or steeper. Thus relatively dense contour lines are seen at the middle slope of gravity deformation.

Flanks of landslides / gravity deformation leave specific shapes in elevation contour lines, such as shown in Figure 2-6 (b). Sudden bends or disturbed shapes are seen in the contour lines with constant density and smooth shape. Such bends or disturbed shapes aligns vertically; such bends and disturbance are seen at the same places in the neighboring contour lines. These sudden bends and disturbance indicate flanks of landslides / gravity deformation.

At the flanks of subsiding areas of landslides / gravity deformation, contour lines show sharp bends toward hill side when it is traced from outside of landslides /gravity deformation, as shown in figure E of Figure 2-6 (b). On the other hand, at the flanks of uplifting areas, contour lines show sharp bends toward valley side as shown in figure B of Figure 2-6 (b).



(a) Term of topo features of landslide with landslide clay



(b) Contour lines showing head scarp/flanks of landslide/gravity deformation

Figure 2-6 Abnormal Contour Lines of Flanks of Landslide / Gravity Deformation

Source: Left: Verne, Right: JICA Project Team

In Figure 2-7, the process of gravity deformation to deep seated landslide is illustrated by contour lines, in which an intact slope is gradually relaxed and deformed by gravity and then collapsed in a form of deep-seated landslide triggered by small failures at the toe of the slope.

Generally, gravity deformation forms tensile zone in the upper part and compressed zone in the lower part of slopes. In the initial stage, the deformation is remarkable in the tensile zone. Thus toppling and up-hill facing scarplets are observed in the upper part of slope, especially at the gentle slope above the shoulder of slope. As the deformation proceeds, bulging at the lower part of the slope appears accompanied by buckling and loosening of bedrock. The bulging slope becomes steeper and small collapse occurs at the lower part, especially at the toe of the slope. Such small collapse significantly destabilize a slope with gravity deformation, eventually induce a deep-seated catastrophic landslides.

In the process of development of gravity deformation from intact slope to deep seated landslide, the anomaly of elevation contours at the stage (c) in In Figure 2-7 is easily detected by map reading.

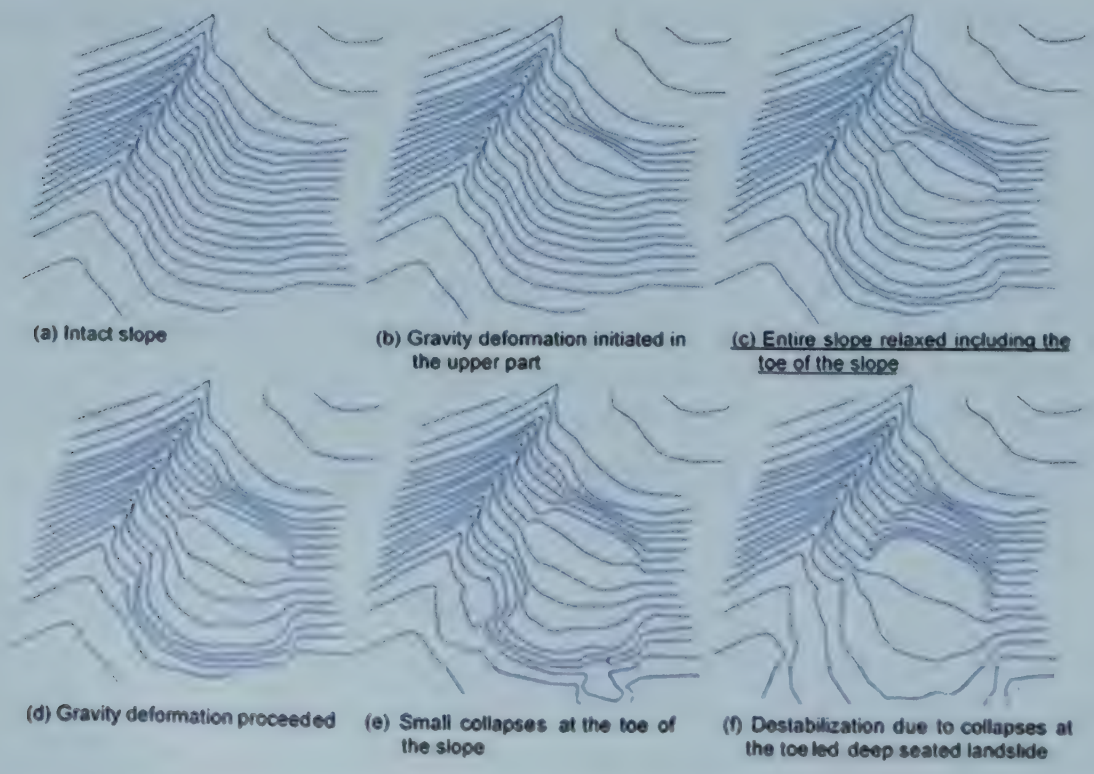


Figure 2-7 Typical Landform Change by Gravity Deformation Seen in Elevation Contours

Source: JICA Project Team

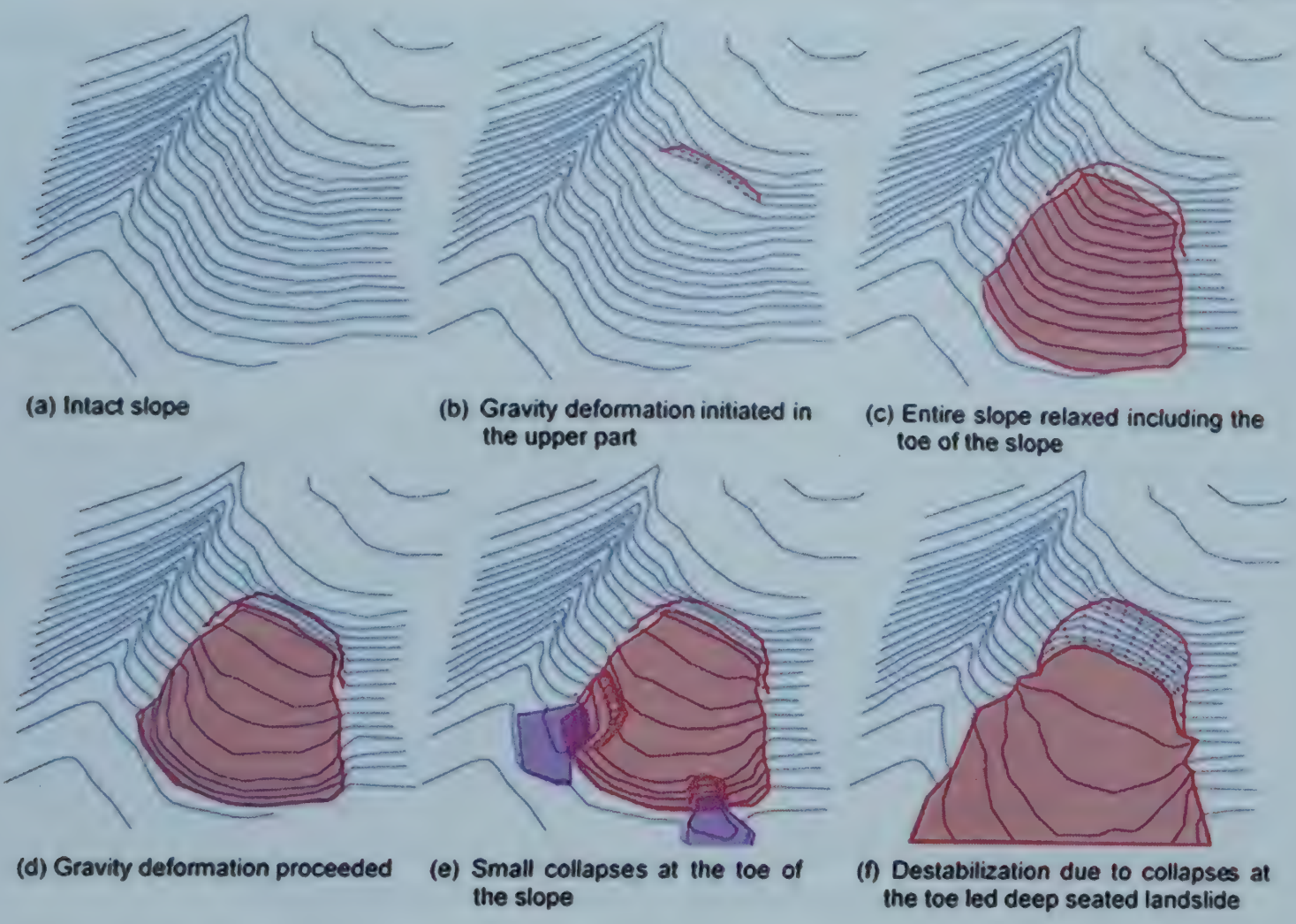


Figure 2-8 Map Reading Results Overlaid on Elevation Contours of Figure 2-7

Source: JICA Project Team

2.2 Exercise of Map Reading

2.2.1 General

The easiest way to learn map reading is to try it repeatedly. In this section, therefore, in addition to the blank maps prepared for subjects of exercise, results of map reading prepared by the authors are attached. In the results of map reading, extracted geomorphological factors such as ridges, valleys, steep slopes, landslides / gravity deformation, debris flow deposits, and talus deposits are illustrated.

The exercise should be done in the following steps:

1 To observe the results by the authors carefully,
2 To copy each geomorphological factor shown in the results by the authors to the blank maps, with pencils,
3 While copying such factors, to understand the features on the contour lines which are focused to identify such factors,
4 While copying such factors, to pay attention to slight changes in curvature of each contour line,
5 While copying such factors, to rotate the blank maps as well as the results by the authors to understand the change in impression of contour lines provided by the rotation so that you could notice new features of the targeted factors and related contour lines,
6 By repeating the process of Step 1 ~5, to improve ability of map reading and interpretation of geomorphological factors,
7 After completing the copying exercise, to try to read the blank maps for the subjects of exercise by yourself without referring to the results by the authors.
8 To check if there are any differences or missing between the results by you in Step 7 and the results by the authors, and

9 While checking the difference in Step 8, to consider and examine why such factors are formed at the place where they are.

By considering and examining the reasons why such geomorphological factors are formed at such places, your understanding on formation of geomorphological factors should be enhanced. To master map reading, it is important to repeat considering and examining the logical reasons and backgrounds for formation of geomorphological factors. In other words, you should keep it in your mind that interpretation through map reading is a repetitive work of self-questioning and self-answering.

Map reading should not end in the aforementioned desk work, however. To verify the interpretation through map reading, we should go to the actual sites where we conduct map reading.

a. In the verifying on site, we should focus the following features of the actual slopes:
b. To verify whether extracted geomorphological factors exist or not.
c. To find out cracks at the shoulder, head, flanks, and toe of the landslides / gravity deformation blocks extracted in map reading
d. To confirm subsiding and uplifting in the said landslides / gravity deformation blocks
e. To check rock fall and small collapse in and around the said landslides / gravity deformation blocks
f. To estimate stability of the said landslides / gravity deformation blocks
g. To estimate risk of the said landslides / gravity deformation blocks

To estimate stability and risk of the landslides / gravity deformation blocks extracted in map reading, confirming related phenomena such as rock fall and small collapse are essential. Checking freshness of cracks at flanks and scarps at shoulders provides clues for activities of landslides / gravity deformation.

By repeating consideration and exercise in desk work and verification and estimation on the actual site, you can master interpretation through map reading and will be able to utilize 2D topographic elevation contour maps as 4D data base with 3D spatial information with time scale.

From the next section, exercise for map reading starts.

As materials for exercise, a blank map and corresponding 3d view are prepared. Figure 2-9 shows a blank map for the subject of exercise. Figure 2-10 shows a bird's-eye view of the actual terrain of the subject of exercise acquired from Google Earth.

In this area, landform partially reflects geological structure underground; the forms of terrain affected by dip slope and plunging slope structures show different gradient and appearance of slope.

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation

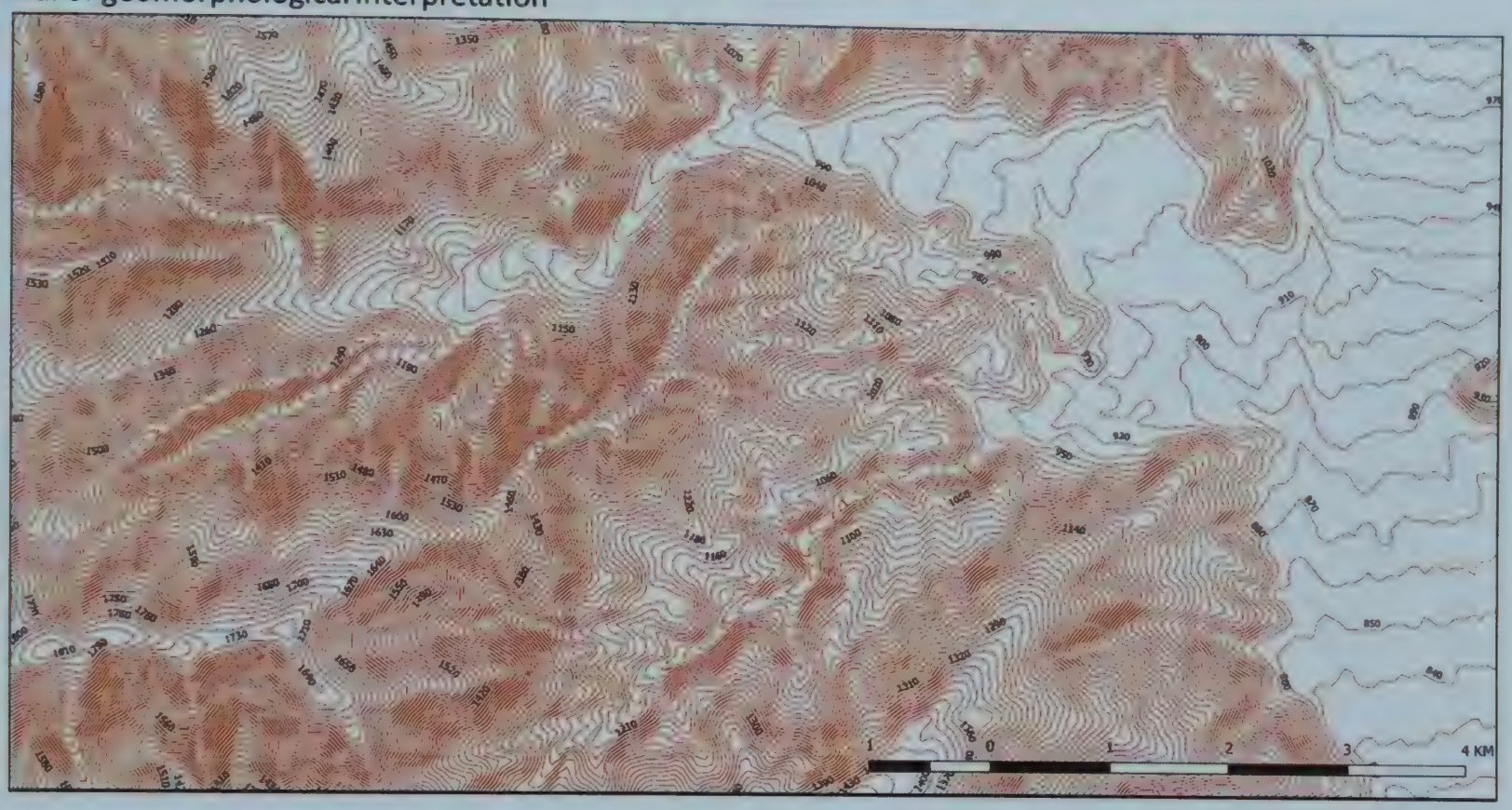


Figure 2-9 Blank Map for Exercise

Source: JICA Project Team

【Practice】 Let's find landslide & debris flow prone area!

Geomorphological features of the debris flow area

Figure 2-10 Bird's-eye View of Actual Terrain for Exercise

Source: Acquired from Google Earth

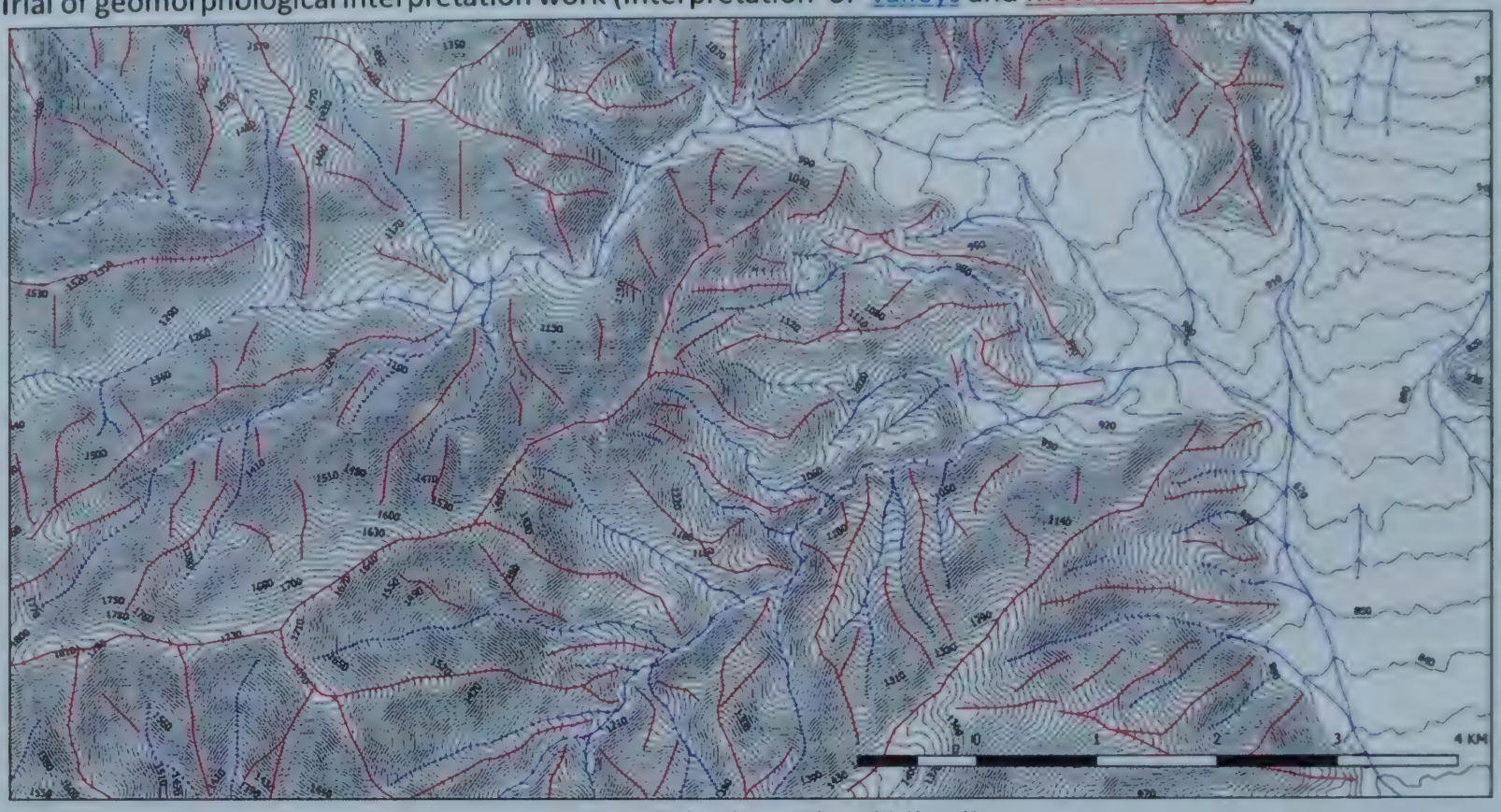
2.2.2 Exercises for Extracting Ridges and Valleys

When extracting ridges or valleys, it is not always necessary to continuously connect the lines of ridges or valleys. Lines of ridges or valleys should be drawn only when it is clearly interpreted. Where it is not clearly interpreted, the lines of ridges or valleys should not be drawn and such lines should end within the areas where clearly interpreted. Disconnection of ridge or

valley lines is sometimes caused by faults or landslide blocks. Thus drawing lines as interpreted as well as considering cause of disconnection from a geological point of view are necessary manner in map reading.

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of valleys and mountain ridges)



Red: Ridge line, Blue: Valley line

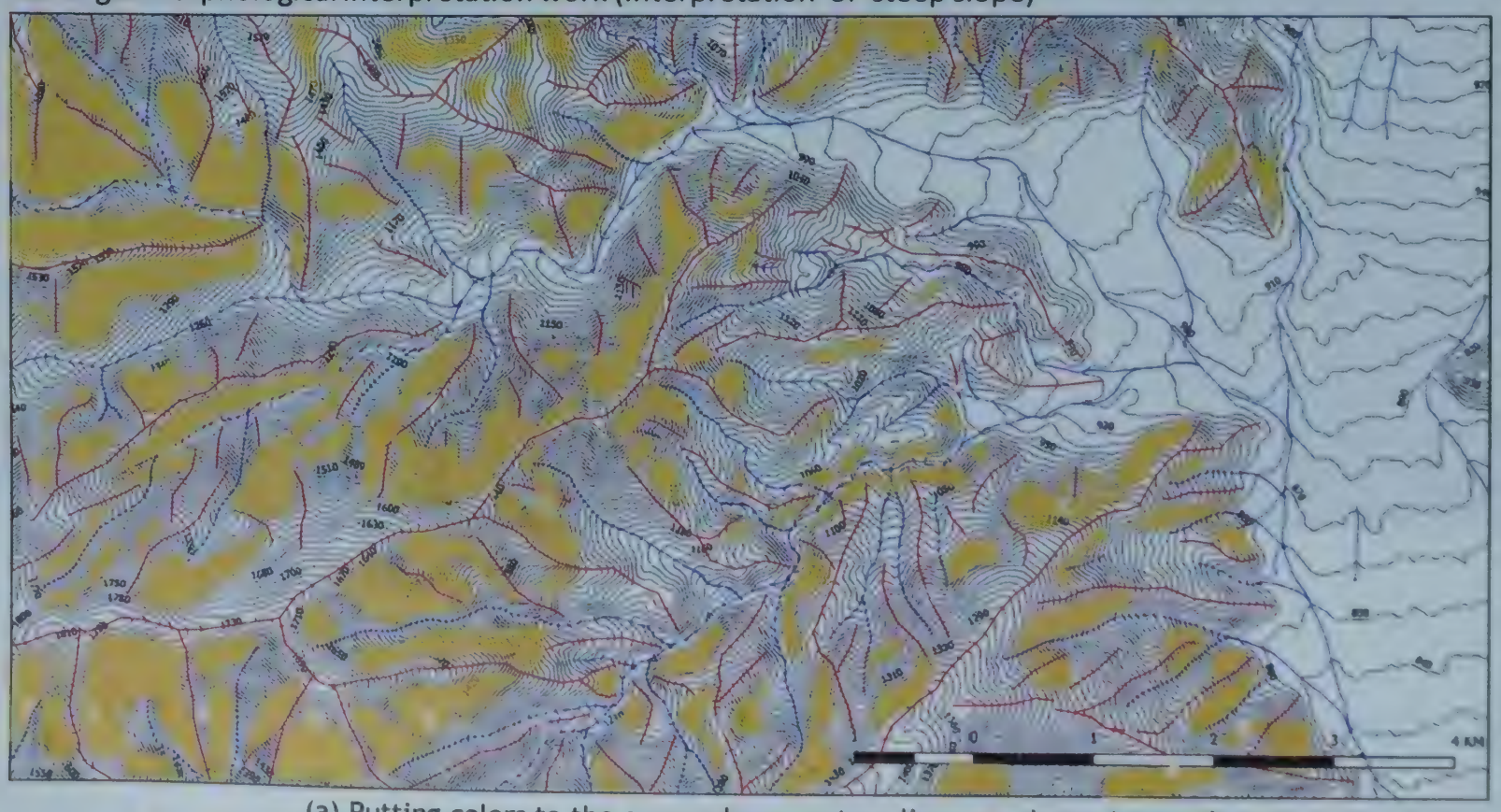
Figure 2-11 Results by Authors: Extracted Ridges and Valleys

Source: JICA Project Team

2.2.3 Exercises for Extracting Steep Slopes

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of steep slope)



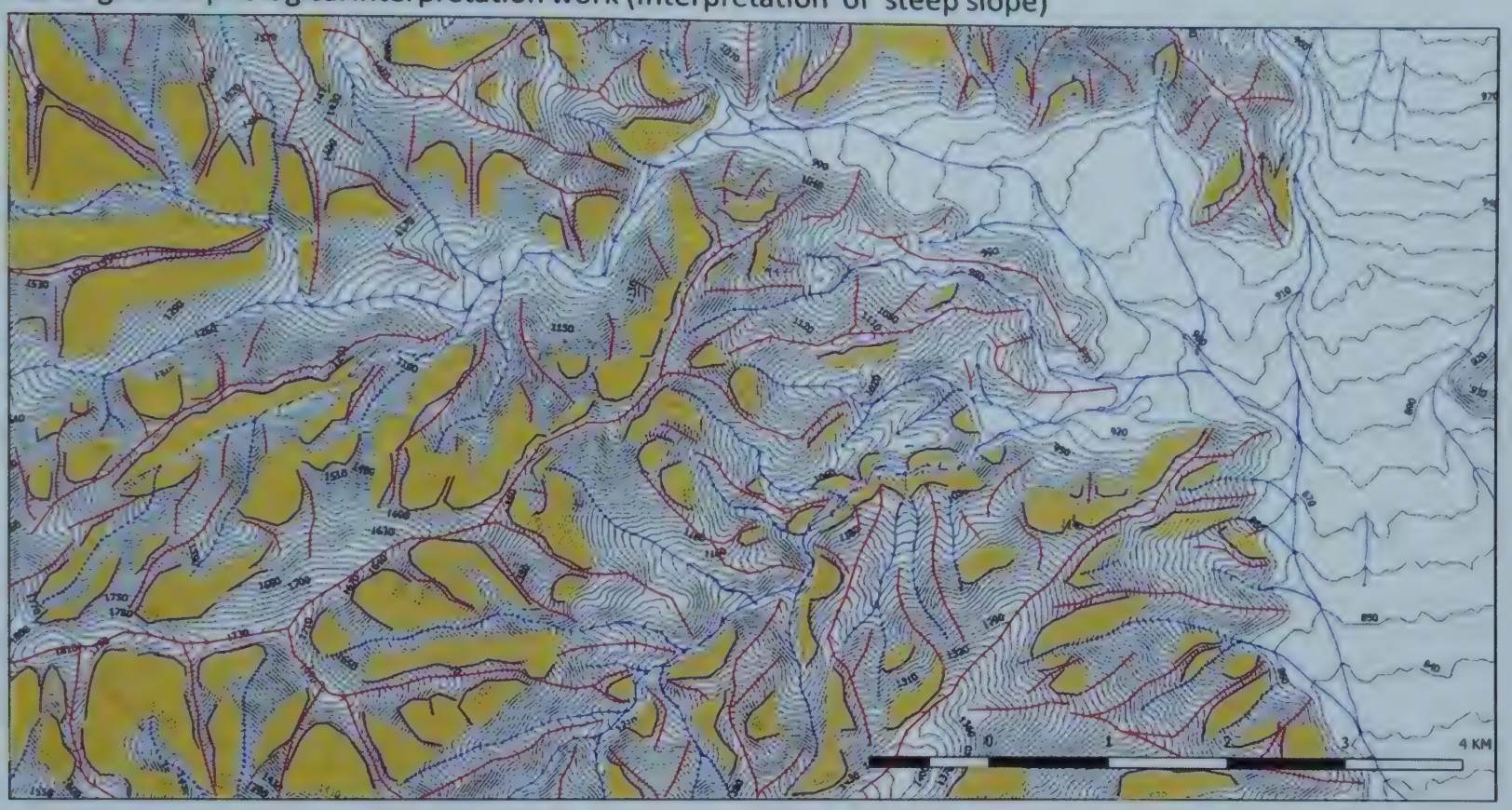
(a) Putting colors to the areas where contour lines are dense (steep slopes)

Figure 2-12 Results by Authors: Extracted Steep Slopes (1)

Source: JICA Project Team

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of steep slope)



(b) Drawing the upper edge of the colored areas of steep slopes

Figure 2-13 Results by Authors: Extracted Steep Slopes (2)

Source: JICA Project Team

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of steep slope)

(c) Removing color then shoulders of steep slopes are stressed

Figure 2-14 Results by Authors: Extracted Steep Slopes (3)

Source: JICA Project Team

Extracting steep slopes is made in the three steps as shown in Figure 2-12 to Figure 2-14, on the map ridge and valley lines are extracted.

(a) Putting colors to the areas where contour lines are dense (steep slopes)
(b) Drawing the upper edge of the colored areas of steep slopes
(c) Removing color then shoulders of steep slopes are stressed

After you are accustomed to it, you may start from step (c) in Figure 2-14.

The term "steep slope" is defined in Japan as slopes with a gradient of 30 degrees or steeper with a vertical height of 5 meters or higher. When extracting steep slope in map reading, sticking to such definition is not necessary. Rather than it, simply marking the areas where contour lines are relatively dense is important.

Considering formation of steep slopes at such places where they are is important when extracting steep slopes. Thus, marking of steep slopes should be placed in relation to other geomorphological features.

2.2.4 Exercises for Extracting Landslides and Gravity Deformation

Extracting landslides and gravity deformation blocks is made on the map steep slopes are extracted, while the symbols of shoulder of steep slopes are partially modified to the symbols of head scarps or erased if needed.

Considering the steep gradients of slopes in the area, most of the blocks extracted in Figure 2-15 are supposed to be gravity deformation. Verifications on whether such blocks are gravity deformation or slow-moving landslide with landslide clay should be made on site by field survey.

On the other hand, blocks with small collapse at its toe may cause large displacements in near future; priorities should be given to such blocks when investigations and mitigation measures are considered.

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of slope deformation)

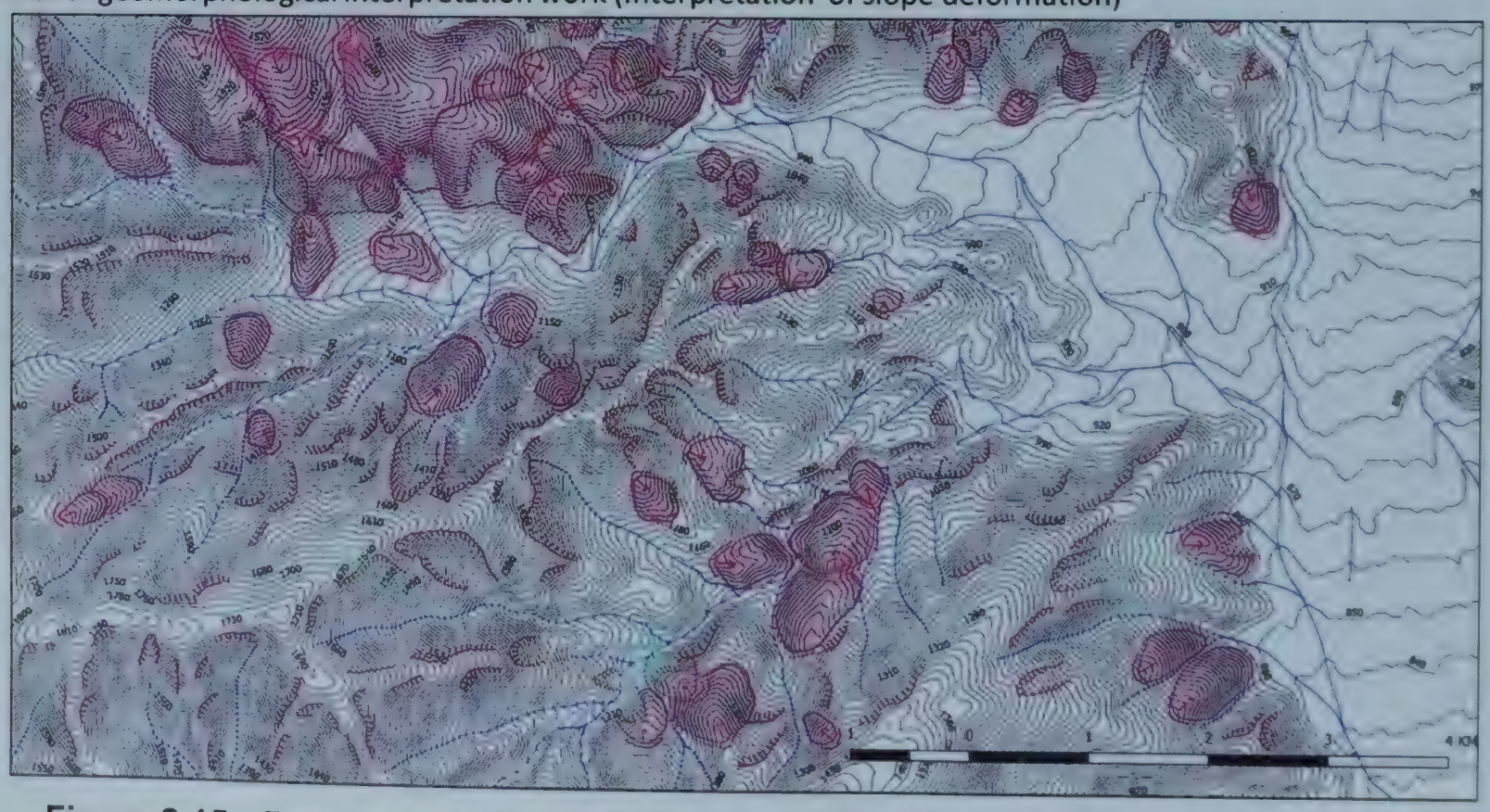


Figure 2-15 Results by Authors: Extracting Landslides and Gravity Deformation

Source: JICA Project Team

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of slope deformation)

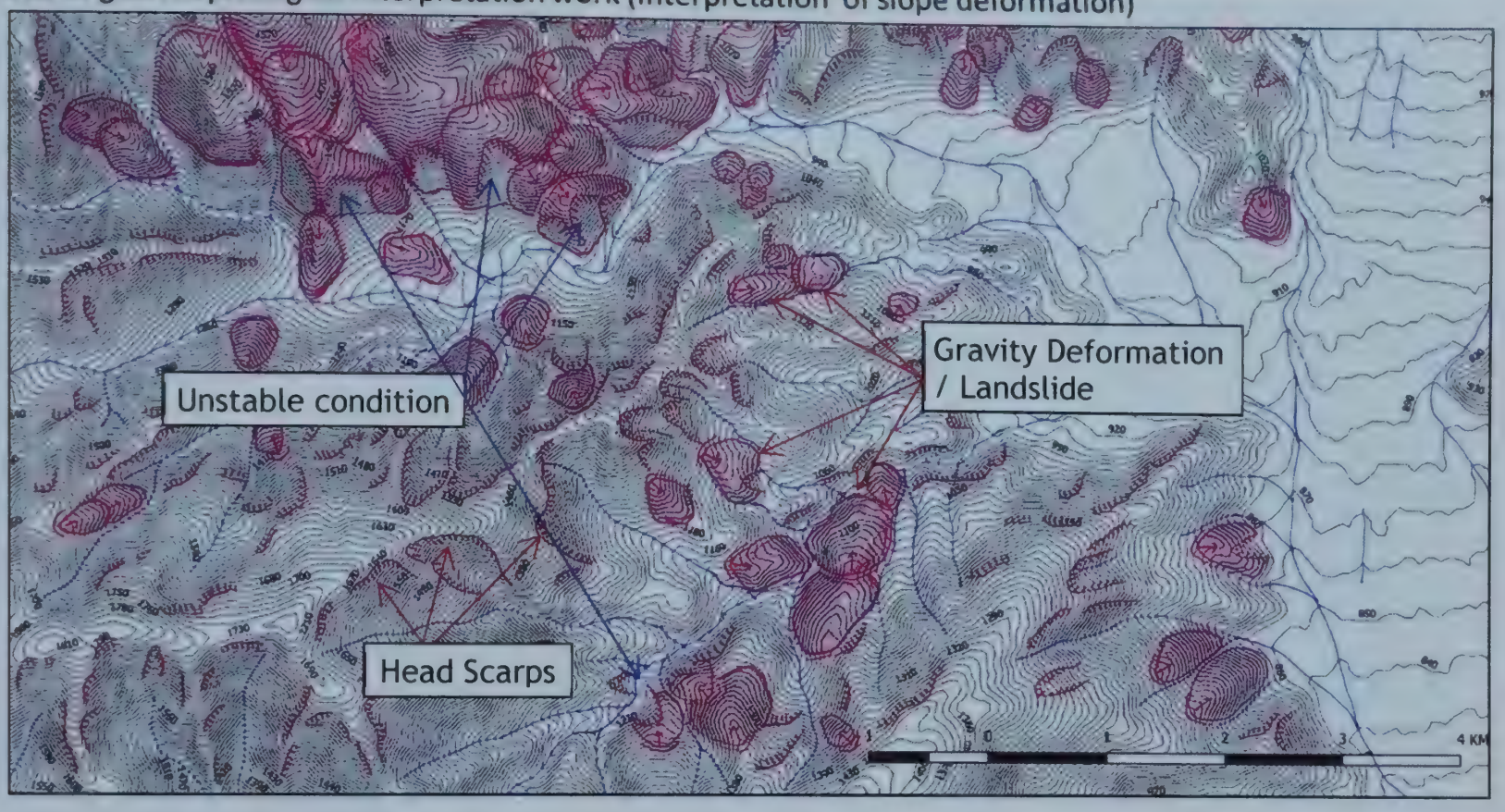


Figure 2-16 Extracting Landslides and Gravity Deformation with Explanation

Source: JICA Project Team

2.2.5 Exercises for Extracting Debris Flow Deposits and Talus

Extracting debris flow deposits and talus is made on the map landslides and gravity deformation are extracted.

Accumulated talus at foot of hill slopes and deposited debris on river beds forms relatively gentle slope. Thus contour lines show wider space from the neighboring ones.

If debris flow deposits and talus are extracted, such streams / slopes should be considered as those with active debris production; susceptible areas of landslides.

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of sediments situation)

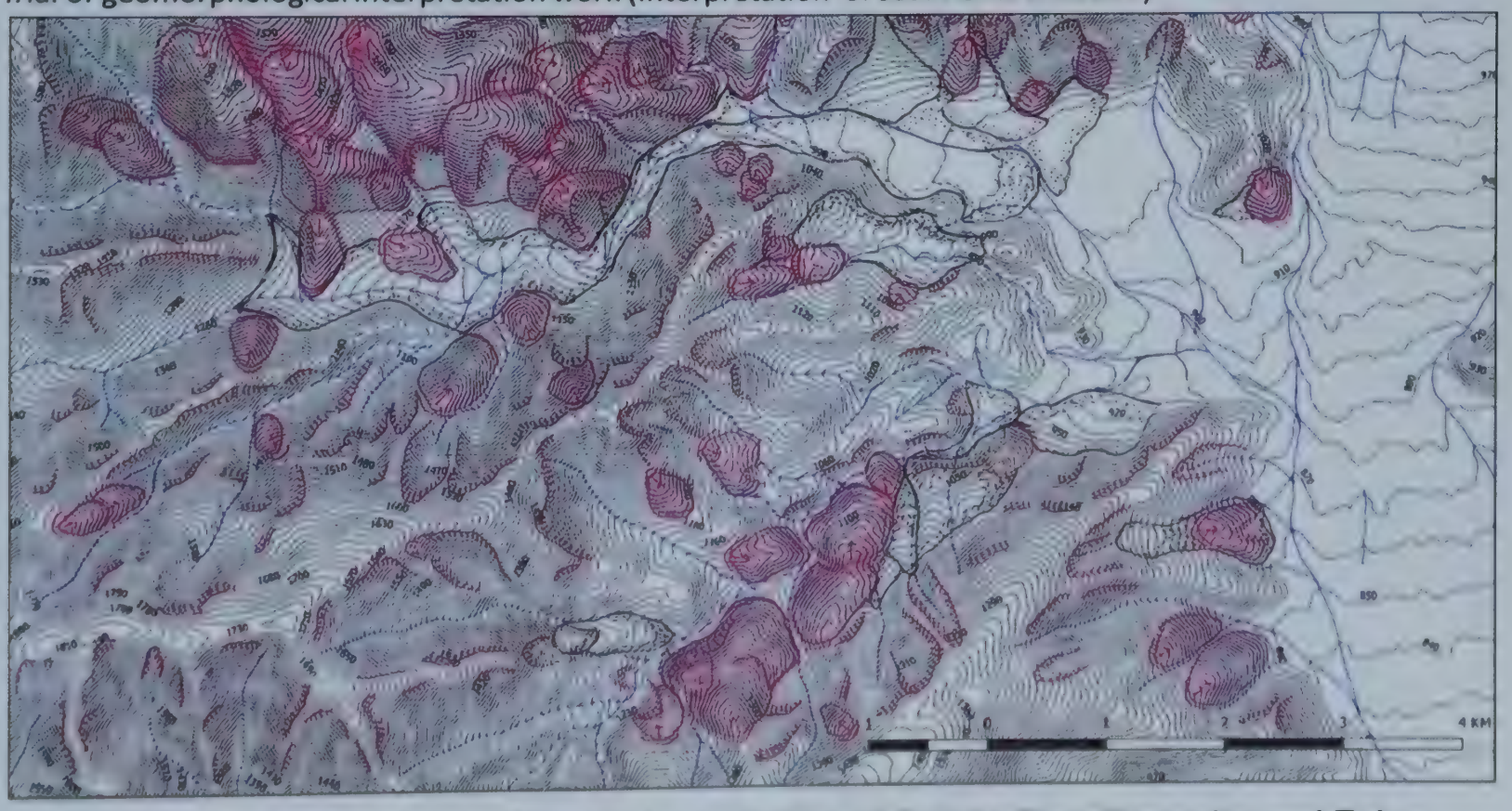


Figure 2-17 Results by Authors: Interpretation of Debris Flow Deposits and Talus

Source: JICA Project Team

【Practice】 Let's find slope failure, gravity deformation & debris flow prone area!

Trial of geomorphological interpretation work (Interpretation of sediments situation)

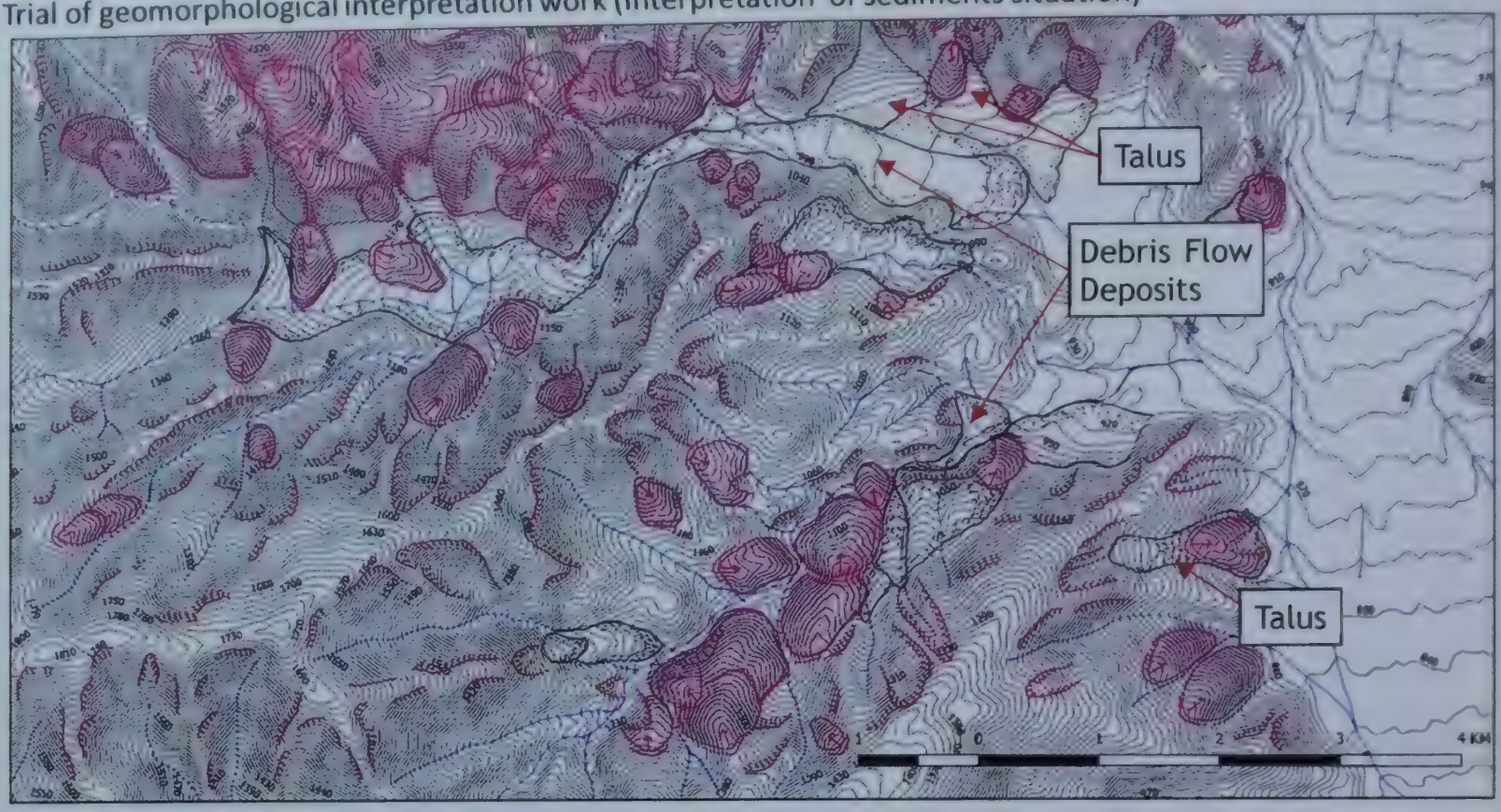


Figure 2-18 Interpretation of Debris Flow Deposits and Talus with Explanation

Source: JICA Project Team

Appendix VI

Technical Specification for Reference:

Cast-in-situ Concrete Crib Works (for EPC)

Cast-in-situ Concrete Crib Works (for Item Rate)

Horizontal Drainage Boring (for EPC)

Horizontal Drainage Boring (for Item Rate)

Drainage Well (for EPC)

Drainage Well (for Item Rate)

3010 CAST-IN-SITU CONCRETE CRIB WORKS (For EPC)

3010.1 Description

This work shall consist of construction of cast in-situ concrete crib works on the slopes. The crib works is a structure aimed at stabilizing the slope by creating a continuous lattice frame on cutting surfaces or natural slopes. The work shall be carried out in accordance with these Specifications and lines, levels and grades, dimensions and cross-sections shown in the Drawings or as directed by the Engineer.

3010.2 Design

(a) General

The concrete crib works shall be so designed through investigation of the site that the works should conform to the conditions of the targeted slope and duly exercise its function.

In case the crib works are intended to work as bearing members of rock bolts or ground anchors, relevant consideration including slope stability analysis shall be done.

In case the crib works are intended to work against earth pressure, relevant calculation shall be executed.

(b) Load

The following loads shall be considered in the design:

- The own weight of the crib works including filling material within the frames and possible accumulation of snow and ice, if applicable.
- The resistance force to be loaded by ground anchors or rock bolts,
- The load from the ground reaction force.

The design load shall be calculated by multiplying working load by load coefficient.

(c) Verification

Design of the crib works shall be verified on the following items:

- Bending moment under ultimate limit state
- Shearing stress under ultimate limit state
- Cracks induced by bending under serviceability limit state
- Cracks induced by shearing stress under serviceability limit state

3010.3 Materials

Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Authority's Engineer.

(a) Cement

Cement shall be Ordinary Portland Cement and conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.

(b) Aggregate
Aggregate shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.
Fine aggregate shall have dry density of 2.5 g/cm^3 or more and a water absorption rate of 3.5% or less.

(c) Admixture
Aggregate shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH or JIS A 6204 or equivalent.

(d) Reinforcement bars
Reinforcing bars shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH or BS4449 Grade 460 or JIS G 3112 or equivalent.

(e) Water
Water shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.

(f) Wire net (or Rath Net)
Wire nets shall conform to JIS G 3552 or effective standards for rhombus shaped chain link wire netting or equivalent. Galvanized coatings for wire nets shall conform to JIS H 8641 HDZ 45 or ISO1461:1999 Steel ≥ 3 mm to < 6mm or equivalent.
Wire nets shall meet the requirements in the following table.

Requirement of Wire Net

Diameter of Wire (mm)	Mesh Size (mm)	Width (m)	Length (m)	Unit Weight (kg/m^2)	Zinc Coating (g/m^2)	Tensile Strength (N/mm^2)
φ2.0	50×50	1.0	10.0	1.0	Not less than 23	290～540

(g) Anchor Pins / Anchor Bars
Anchor bars and anchor pins shall conform to BS 4449 Grade 460 or equivalent. Galvanized coatings for anchor pins and anchor bars shall conform to JIS H 8641 HDZ 45 or ISO1461:1999 Steel ≥ 3 mm to < 6mm or equivalent.

(h) Soil
Soil use for necessary filling shall not be detrimental for vegetation growth and shall be to the satisfaction of the Engineer.

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings to the Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Engineer has the authority to relax or do away with this requirement, if he chooses to do so, on a written request made by the Contractor.

The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Preparation of Slopes

(i) Trimming and clearing
The Contractor shall trim and clear a slope by hand and/or construction machinery. Where water is encountered due to seepage, springs, and other reasons on a slope, the Contractor shall take adequate measures after reporting to the Engineer for his direction. The Contractor shall notify the Engineer of the completion of slope trimming and clearing for his approval.

(ii) Removal of Rock and Boulders
Unstable rocks on a slope shall be removed by hand as much as possible. Unstable rocks to be removed shall be selected and reported to the Engineer for his approval.

(iii) Removal of trees and shrubs
Trees and shrubs which may obstruct the work shall be removed prior to the commencement of the works. Trees and shrubs to be removed shall be selected by the Contractor and approved by the Engineer.

(iv) Disposal of surplus soil and removed trees and shrubs
Disposal method of surplus soil and removal of trees and shrubs shall conform to laws and regulation, the Specifications or the Engineer's direction

(v) Payment for slope trimming and clearing, removal of surplus soil, boulders, and rock, removal of trees and shrubs, disposal of surplus soil, rock, and boulders, and disposal of trees and shrubs shall be deemed to be included in the unit rates for crib works.

(c) Treatment of Uneven Surface to Place Frames of Crib Works

Uneven surface of the slopes where frames of crib works are placed shall be so treated that the frames of crib works can closely contact with the surface of the slope.

Unevenness shall be treated by a methods satisfactory to the Engineer. Spraying mortar using a shotcreting machine may be used to ease unevenness of the surface to place frames, excluding the areas within the frames unless shotcreting is specified in the drawings for such areas.

Unless otherwise indicated in this Specification, mortar shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH and the following table:

Mortal for Treatment for Unevenness

Type of work	Design Strength	Unit Cement Content	Cement-Sand Proportion	Water-Cement Ratio
Shotcrete works	18 N/mm2	Not less than 400 kg/cm3	Cement : Sand = 1 : 4	Not more than 60 %

The consistency of the mortar shall be suitable to be used with shotcreting machine.

Payment for smoothing the surface shall be deemed to be included in the unit rates for crib work.

(d) Fixing Reinforcing Bars, Anchor Bars, Anchor Pins, and Wire Net, and Placing Temporary Formwork

Wire nets shall be spread over and firmly fixed to a slope, including the areas within the frames of crib works. Reinforcing bars, anchor bars, and anchor pins as well as temporary formwork shall be arranged and firmly fixed to the ground to avoid any vibration and displacement until the concrete to be casted into frames becomes hardened. The Contractor shall notify the Engineer of the completion of arrangements of reinforcing bars and wire nets as well as formwork for his approval.

Payment for smoothing the surface by mortar, arrangement of wire nets, reinforcing bars, anchor bars, anchor pins and temporary formwork shall be deemed to be included in the unit rates for crib work.

(e) Casting Concrete to Crib Frames

Unless otherwise indicated in this Specification, concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH.

The concrete used in the work shall conform to the following table and shall have suitable consistency for pneumatic transport and casting:

Grade of Concrete

Type of work	Type of concrete	Grade of concrete	Max. size of aggregate (mm)
Cast-in-situ concrete crib works	Standard	M20	20

Curing of the concrete cast in frames shall be duly performed. Joint of concrete shall be adequately cleaned so as to prevent cold joints. Joints shall not be placed at vertical frames. Casting of concrete for upper frames should be done only after strength of concrete for lower frames achieves enough strength to support the weight of the upper frames. After enough period for curing is secured, temporary formwork shall be removed and the Contractor shall clear the area within frames.

(f) Filling with Soil above the Horizontal Beams of the Crib

Water may collect and pool in prismatic sections above the horizontal beams of the crib. In order to prevent this, and to facilitate the surface water to easily drain off, the prismatic sections shall be filled with soil which is not detrimental to vegetation growth.

No payment will be made for this work and shall be deemed to be included in the unit rates.

(g) Treatment for the Areas within Frames

The Contractor shall perform the required works for the areas within the frames specified in the drawings.

3010.5 Record

(a) Record

The necessary data for maintenance and management shall be recorded and preserved.

DRAFT

3010 CAST-IN-SITU CONCRETE CRIB WORKS (For Item Rate)

3010.1 Description

This work shall consist of construction of cast in-situ concrete crib works on the slopes. The crib works is a structure aimed at stabilizing the slope by creating a continuous lattice frame on cutting surfaces or natural slopes. The work shall be carried out in accordance with these Specifications and lines, levels and grades, dimensions and cross-sections shown in the Drawings or as directed by the Engineer.

3010.2 Materials

Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Authority's Engineer.

(a) Cement

Cement shall be Ordinary Portland Cement and conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.

(b) Aggregate

Aggregate shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.

Fine aggregate shall have dry density of 2.5 g/cm^3 or more and a water absorption rate of 3.5% or less.

(c) Admixture

Aggregate shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH or JIS A 6204 or equivalent.

(d) Reinforcement bars

Reinforcing bars shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH or BS4449 Grade 460 or JIS G 3112 or equivalent.

(e) Water

Water shall conform to Section 1000 of the Specifications for Road and Bridge Works of MoRTH.

(f) Wire net (or Rath Net)

Wire nets shall conform to JIS G 3552 or effective standards for rhombus shaped chain link wire netting or equivalent. Galvanized coatings for wire nets shall conform to JIS H 8641 HDZ 45 or ISO1461:1999 Steel ≥ 3 mm to < 6mm or equivalent.
Wire nets shall meet the requirements in the following table.

Requirement of Wire Net

Diameter of Wire (mm)	Mesh Size (mm)	Width (m)	Length (m)	Unit Weight (kg/m^2)	Zinc Coating (g/m^2)	Tensile Strength (N/mm^2)
φ2.0	50×50	1.0	10.0	1.0	Not less than 23	290～540

(g) Anchor Pins / Anchor Bars
Anchor bars and anchor pins shall conform to BS 4449 Grade 460 or equivalent. Galvanized coatings for anchor pins and anchor bars shall conform to JIS H 8641 HDZ 45 or ISO1461:1999 Steel ≥ 3 mm to < 6mm or equivalent.

(h) Soil
Soil use for necessary filling shall not be detrimental for vegetation growth and shall be to the satisfaction of the Engineer.

3010.3 Construction Requirements

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings to the Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Engineer has the authority to relax or do away with this requirement, if he chooses to do so, on a written request made by the Contractor.

The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Preparation of Slopes

(i) Trimming and clearing
The Contractor shall trim and clear a slope by hand and/or construction machinery. Where water is encountered due to seepage, springs, and other reasons on a slope, the Contractor shall take adequate measures after reporting to the Engineer for his direction. The Contractor shall notify the Engineer of the completion of slope trimming and clearing for his approval.

(ii) Removal of Rock and Boulders
Unstable rocks on a slope shall be removed by hand as much as possible. Unstable rocks to be removed shall be selected and reported to the Engineer for his approval.

(iii) Removal of trees and shrubs
Trees and shrubs which may obstruct the work shall be removed prior to the commencement of the works. Trees and shrubs to be removed shall be selected by the Contractor and approved by the Engineer.

(iv) Disposal of surplus soil and removed trees and shrubs
Disposal method of surplus soil and removal of trees and shrubs shall conform to laws and regulation, the Specifications or the Engineer's direction

(v) Payment for slope trimming and clearing, removal of surplus soil, boulders, and rock, removal of trees and shrubs, disposal of surplus soil, rock, and boulders, and disposal of trees and shrubs shall be deemed to be included in the unit rates for crib works.

(c) Treatment of Uneven Surface to Place Frames of Crib Works
Uneven surface of the slopes where frames of crib works are placed shall be so treated that the frames of crib works can closely contact with the surface of the slope.
Unevenness shall be treated by a methods satisfactory to the Engineer. Spraying mortar using a shotcreting machine may be used to ease unevenness of the surface to place frames, excluding the areas within the frames unless shotcreting is specified in the drawings for such areas.
Unless otherwise indicated in this Specification, mortar shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH and the following table:

Mortal for Treatment for Unevenness

Type of work	Design Strength	Unit Cement Content	Cement-Sand Proportion	Water-Cement Ratio
Shotcrete works	18 N/mm2	Not less than 400 kg/cm3	Cement : Sand = 1 : 4	Not more than 60 %

The consistency of the mortar shall be suitable to be used with shotcreting machine.
Payment for smoothing the surface shall be deemed to be included in the unit rates for crib work.

(d) Fixing Reinforcing Bars, Anchor Bars, Anchor Pins, and Wire Net, and Placing Temporary Formwork

Wire nets shall be spread over and firmly fixed to a slope, including the areas within the frames of crib works. Reinforcing bars, anchor bars, and anchor pins as well as temporary formwork shall be arranged and firmly fixed to the ground to avoid any vibration and displacement until the concrete to be casted into frames becomes hardened. The Contractor shall notify the Engineer of the completion of arrangements of reinforcing bars and wire nets as well as formwork for his approval.

Payment for smoothing the surface by mortar, arrangement of wire nets, reinforcing bars, anchor bars, anchor pins and temporary formwork shall be deemed to be included in the unit rates for crib work.

(e) Casting Concrete to Crib Frames

Unless otherwise indicated in this Specification, concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH.

The concrete used in the work shall conform to the following table and shall have suitable consistency for pneumatic transport and casting:

Grade of Concrete

Type of work	Type of concrete	Grade of concrete	Max. size of aggregate (mm)
Cast-in-situ concrete crib works	Standard	M20	20

Curing of the concrete cast in frames shall be duly performed. Joint of concrete shall be adequately cleaned so as to prevent cold joints. Joints shall not be placed at vertical frames. Casting of concrete for upper frames should be done only after strength of concrete for lower frames achieves enough strength to support the weight of the upper frames. After enough period for curing is secured, temporary formwork shall be removed and the Contractor shall clear the area within frames.

(f) Filling with Soil above the Horizontal Beams of the Crib

Water may collect and pool in prismatic sections above the horizontal beams of the crib. In order to prevent this, and to facilitate the surface water to easily drain off, the prismatic sections shall be filled with soil which is not detrimental to vegetation growth.

No payment will be made for this work and shall be deemed to be included in the unit rates.

(g) Treatment for the Areas within Frames

The Contractor shall perform the required works for the areas within the frames specified in the drawings.

3010.4 Record

(a) Record
The necessary data for maintenance and management shall be recorded and preserved.

3010.5 Measurement and Payment

(a) Measurement
The crib shall be measured in linear metres along each row. The lengths of each row shall be added to arrive at the final quantity of the crib.

(b) Payment
The quantity of crib as measured above shall be paid in the Contract unit rate of linear meters. The rates shall include all that has to be carried out to place the crib in place. The rate shall be full compensation for all labour, material, equipment and incidentals required to complete the work.

Pay Items and Pay Units shall be:

Pay Item	Description	Pay Unit
3010(1)	Cast-in-situ Concrete Cribs Works	Linear Meter

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3011 Horizontal Drainage Borings (For EPC)

3011.1 Description

This work shall consist of drilling holes which are inclined to the horizontal direction by not more than 10 degrees and providing and installing perforated pipes and protective pipes into the holes so drilled. The work shall be performed in open space or inside the drainage well as the case may be. The work shall be carried out in accordance with these Specifications and lines, levels and grades, dimensions and cross-sections shown in the Drawings or as directed by the Engineer.

3011.2 Design

(a) General

The horizontal drainage borings shall be so designed through investigation of the site that the works should conform to the conditions of the targeted slope and dully exercise its function.

(b) Purpose

The purpose of the works is to drain ground water above slip surface so that pore water pressure affecting slip surface can be reduced. Arrangement of drainage pipe shall be, therefore, so planed that pore water pressure should be reduced to a maximum extent. Drainage pipe shall have, therefore, enough length to penetrate slip surface.
The works are also planned to ease effects of seepage pressure of unconfined groundwater.

(c) Topographic Survey

Topographic survey shall be so executed that plan view and profile of landslides could be prepared.

(d) Investigation

Investigation to understand the depth and extent of slip surface as well as groundwater level shall be executed prior to design of the works. Borehole investigations at the targeted landslides shall be done while obtaining core samples, performing in-situ tests, and installing monitoring wells. Borehole investigation shall be so planned that slip surface and ground water level should be understood as much as possible.

(e) Necessary Length of Borings

<u>To ease pore water pressure</u>

- A section of 5 meters or more shall be secured after penetrating the deepest slip surface.

<u>To ease seepage pressure of unconfined groundwater</u>

- In debris or colluvium or terrace deposit: more than 25 meters
- In loosened zone of bedrock: more than 40 meters

(f) Arrangements of Borings

Horizontal drainage borings shall be arranged in the following manner:

- The borings shall be arranged parallel or in a fan shape.
- The ends in the ground of drainage borings shall have a horizontal distance of 5 meters or more from the neighboring borings.
- The borings shall have an angle of elevation between 5 and 10 degrees from the horizontal level.

(g) Effect on Slope Stability Analysis
As the effects of the works, reduction of groundwater level in slope stability analysis shall be not more than 3 meters deep.

(h) Drainage System to Lead Drained Water to the Nearest Stream or Drainage Ditches
Along with horizontal drainage borings, drainage system shall be so designed that groundwater flowing from the outlet of the borings is duly collected and lead to the nearest stream or drainage ditches which has enough capacity to accommodate additional flow from the drainage borings.

3011.3 Materials
Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Authority's Engineer.

(a) Perforated Pipes
Perforated pipes shall be vinyl chloride pipes (VP) and shall conform to the following requirements and JIS K 6741 or ISO 3633:2002 or an equivalent standard:

Nominal Diameter	External Diameter	Tolerance of External Diameter	Thickness	Tolerance of Thickness
50mm	60mm	±0.4 mm	4.1 mm	+ 0.8 mm

The pipes shall have provisions for male-female joints conforming to JIS K 6739 or ISO 1452-1:2009 & ISO 1452-2:2009 & ISO 3633:2002 & ISO 4435:2003 or an equivalent standard. The sleeve length of the joints shall be at least 50 mm and the thickness of the sleeve shall be 2 mm. The external diameter of the joint shall be the same as the external diameter of the pipe.
The pipes shall be perforated as shown on the Drawings. The Contractor's Method Statement shall show how the perforations will be made.

l is approximately 125 mm
d is approximately 5 mm

The ends placed in the ground of perforated pipes should be so protected by caps or covers that clogging by soil or algae is prevented.

(b) Filter Material
Filter material of geotextile shall conform to the general filter requirements as per Specifications for Road and Bridge Works of MoRTH.

(c) Protective Pipes
The protective pipes shall be PVC pipes and conform to the following requirements:

Nominal Diameter	Nominal Pressure	External Diameter	Internal Diameter
90 mm	Type 400 kPa	90 mm	85 mm
75 mm	Type 400 kPa	75 mm	71 mm

3011.4 Construction Requirements

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings to the Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Engineer has the authority to relax or do away with this requirement, if he chooses to do so, on a written request made by the Contractor.

The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Drilling the holes
The contractor shall clear the face around the drilling location.

The contractor shall use appropriate equipment for drilling the holes, taking into consideration of the factors such as soil properties, ground conditions, design requirements, site conditions etc. The design requirements include, among other things, the diameter of the perforated pipe, the diameter of the protective pipe etc.

Samples of material encountered shall be collected at every 1.0m of the drill hole and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label shall include information such as identification number, location within the drill hole where the sample was collected etc.

Within 14 days of the completion of the drilling of each hole, the Contractor shall submit a drilling report to the Engineer, accurately describing the boundaries of the geological formations etc. The format of the drilling report shall have the prior approval of the Engineer.

The Contractor shall pull out the drilling rods, in the presence of the Engineer and the length of the drilling rods shall be measured by the Engineer and the Contractor, which shall form the basis of deciding the depth of the drill-hole.
The drill holes shall be within the following tolerances:

Category	Tolerance (mm)
Length	Design length or more
Displacement	100.0

Clearing the face around the drilling location, collection of samples and submission of drilling reports shall not be measured nor paid for and deemed to be included in the rate for drilling.

(c) Installation of the perforated pipes

Before the commencement of the installation, the Contractor shall obtain permission from the Engineer to do so.

The perforated section of the pipes should be protected by filter material of geotextile. The filter should be neatly fixed to the pipes by vinyl tapes or wires in a manner any slack or loosening of filter is avoided

The perforated pipes shall be installed into the whole length of the drill hole. If the pipes are to be connected, the connections shall be sturdy so that there will be no loosening of connections. The connections shall be of male-female type and conform to JIS K 6739 or ISO 1452-1:2009 & ISO 1452-2:2009 & ISO 3633:2002 & ISO 4435:2003 or an equivalent standard.

(d) Installation of protective pipe

The protective pipe shall be installed as shown in the Drawings or as instructed by the Engineer. It shall be installed after the completion of the installation of the perforated pipe. The protected pipe shall be placed through the gabion wall and bounded to the gabion boxes as shown in the Drawings.

(e) Gabion Wall

A gabion wall shall be constructed at the outlet of the perforated pipe, as per Specifications for Road and Bridge Works of MoRTH in order to protect the slope at such outlet.

3011.5 Record

(a) Record

The necessary data for maintenance and management shall be recorded and preserved.

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3011 Horizontal Drainage Borings (For Item Rate)

3011.1 Description

This work shall consist of drilling holes which are inclined to the horizontal direction by not more than 10 degrees and providing and installing perforated pipes and protective pipes into the holes so drilled. The work shall be performed in open space or inside the drainage well as the case may be. The work shall be carried out in accordance with these Specifications and lines, levels and grades, dimensions and cross-sections shown in the Drawings or as directed by the Engineer.

3011.2 Materials

Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Authority's Engineer.

(a) Perforated Pipes

Perforated pipes shall be vinyl chloride pipes (VP) and shall conform to the following requirements and JIS K 6741 or ISO 3633:2002 or an equivalent standard:

Nominal Diameter	External Diameter	Tolerance of External Diameter	Thickness	Tolerance of Thickness
50mm	60mm	±0.4 mm	4.1 mm	+ 0.8 mm

The pipes shall have provisions for male-female joints conforming to JIS K 6739 or ISO 1452-1:2009 & ISO 1452-2:2009 & ISO 3633:2002 & ISO 4435:2003 or an equivalent standard. The sleeve length of the joints shall be at least 50 mm and the thickness of the sleeve shall be 2 mm. The external diameter of the joint shall be the same as the external diameter of the pipe.

The pipes shall be perforated as shown on the Drawings. The Contractor's Method Statement shall show how the perforations will be made.

l is approximately 125 mm
d is approximately 5 mm

The ends placed in the ground of perforated pipes should be so protected by caps or covers that clogging by soil or algae is prevented.

(b) Filter Material

Filter material of geotextile shall conform to the general filter requirements as per Specifications for Road and Bridge Works of MoRTH.

(c) Protective Pipes

The protective pipes shall be PVC pipes and conform to the following requirements:

Nominal Diameter	Nominal Pressure	External Diameter	Internal Diameter
90 mm	Type 400 kPa	90 mm	85 mm
75 mm	Type 400 kPa	75 mm	71 mm

3011.3 Construction Requirements

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings to the Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Engineer has the authority to relax or do away with this requirement, if he chooses to do so, on a written request made by the Contractor.

The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Drilling the holes
The contractor shall clear the face around the drilling location.

The contractor shall use appropriate equipment for drilling the holes, taking into consideration of the factors such as soil properties, ground conditions, design requirements, site conditions etc. The design requirements include, among other things, the diameter of the perforated pipe, the diameter of the protective pipe etc.

Samples of material encountered shall be collected at every 1.0m of the drill hole and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label shall include information such as identification number, location within the drill hole where the sample was collected etc.

Within 14 days of the completion of the drilling of each hole, the Contractor shall submit a drilling report to the Engineer, accurately describing the boundaries of the geological formation etc. The format of the drilling report shall have the prior approval of the Engineer.

The Contractor shall pull out the drilling rods, in the presence of the Engineer and the length of the drilling rods shall be measured by the Engineer and the Contractor, which shall form the basis of deciding the depth of the drill-hole.
The drill holes shall be within the following tolerances:

Category	Tolerance (mm)
Length	Design length or more
Displacement	100.0

Clearing the face around the drilling location, collection of samples and submission of drilling reports shall not be measured nor paid for and deemed to be included in the rate for drilling.

(c) Installation of the perforated pipes

Before the commencement of the installation, the Contractor shall obtain permission from the Engineer to do so.

The perforated section of the pipes should be protected by filter material of geotextile. The filter should be neatly fixed to the pipes by vinyl tapes or wires in a manner any slack or loosening of filter is avoided

The perforated pipes shall be installed into the whole length of the drill hole. If the pipes are to be connected, the connections shall be sturdy so that there will be no loosening of connections. The connections shall be of male-female type and conform to JIS K 6739 or ISO 1452-1:2009 & ISO 1452-2:2009 & ISO 3633:2002 & ISO 4435:2003 or an equivalent standard.

(d) Installation of protective pipe

The protective pipe shall be installed as shown in the Drawings or as instructed by the Engineer. It shall be installed after the completion of the installation of the perforated pipe. The protected pipe shall be placed through the gabion wall and bounded to the gabion boxes as shown in the Drawings.

(e) Gabion Wall

A gabion wall shall be constructed at the outlet of the perforated pipe, as per Specifications for Road and Bridge Works of MoRTH in order to protect the slope at such outlet.

3011.4 Record

(a) Record

The necessary data for maintenance and management shall be recorded and preserved.

3011.5 Measurements and Payment

(a) Measurement

The depth of drilling completed is measured in linear meters by the length of the drill rods used for such drilling. The length of drilling through various types of soil / boulders/ rock will be based on the drilling report submitted by the Contractor and accepted by the Engineer.

The perforated pipe shall be measured in linear meters, by the length of pipes installed. The overlap of pipes at joints shall not be measured.

The protective pipe shall be measured in linear meters, by the length of the pipes installed. The overlap of pipes at joints shall not be measured.

(b) Payment

The accepted quantities of the drilling shall be paid for at the Contract unit price per linear meter of the size of the drill-hole. This rate shall include the cost for preparing the face of the drill hole and protection of the outlets.

The accepted quantities of pipe shall be paid for at the Contract unit price per linear meter of the type and size of pipe as specified, complete in place. Cost of providing and installing connections, branch connections, end connections, elbows shall be deemed to be included in the rate of pipes.

The rates shall include all that has to be carried out to drill and place pipes. The rate shall be full compensation for all labour, material, equipment and incidentals required to complete the works.

The Pay Items and Pay Units will be as follows: -

Pay Item	Description	Pay Unit
3011(1)	Drilling in suitable or unsuitable soils or soft rock in open space (state the nominal diameter of pipe to be inserted)	lm
3011(2)	Drilling in boulders or hard rock in open space (state the nominal diameter of pipe to be inserted)	lm
3011(3)	Perforated pipes in open space (state nominal diameter of pipe)	lm
3011(4)	Protective pipes in open space (state nominal diameter of pipe)	lm

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3012 DRAINAGE WELL (For EPC)

3012.1 Description

This work shall consist of constructing wells and providing associated facilities in order to collect water from the land slide / slope failure prone areas and safely discharge the water so collected into natural or man-made canals or drains.

Well excavation shall consist of all the required excavation within the well. The work shall include the removal, stockpiling, handling hauling and proper utilization in the works or disposal to spoil tips, located by the Contractor and approved by the Authority's Engineer, of all excess excavated material.

3011.2 Design

(a) General

The drainage wells shall be so designed through investigation of the site that the works should conform to the conditions of the targeted slope and dully exercise its function.

(b) Purpose

The purpose of the works is to drain ground water above slip surface in deep underground where the horizontal drainage boring from the ground surface is unable to reach so that pore water pressure affecting slip surface in deep underground can be reduced. Arrangement of drainage well, horizontal drainage boring and discharge boring shall be, therefore, so planed that pore water pressure should be reduced to a maximum extent. Drainage pipe of horizontal drainage boring shall have, therefore, enough length to penetrate slip surface at proper depths.

The works are also planned to ease effects of seepage pressure of unconfined groundwater.

(c) Topographic Survey

Topographic survey shall be so executed that plan view and profile of landslides could be prepared.

(d) Investigation

Investigation to understand the depth and extent of slip surface as well as groundwater level shall be executed prior to design of the works. Borehole investigations at the targeted landslides shall be done while obtaining core samples, performing in-situ tests, and installing monitoring wells. Borehole investigation shall be so planned that slip surface and ground water level should be understood as much as possible. The monitoring wells should be used for consecutive observation of groundwater level, which should be started prior to the commencement of construction so that the effects by the drainage wells constructed in the project could be well evaluated.

(e) Necessary Length of Borings

<u>To ease pore water pressure</u>

- A section of 5 meters or more shall be secured after penetrating the deepest slip surface.

<u>To ease seepage pressure of unconfined groundwater</u>

- In debris or colluvium or terrace deposit: more than 25 meters
- In loosened zone of bedrock: more than 40 meters

Necessary Depth of Wells

- To provide working space for horizontal drainage boring to penetrate slip surface in

deep underground. The maximum lengths of horizontal drainage boring practically used is 50 meters. In case the horizontal drainage boring from the ground surface cannot penetrate the slip surface which runs deep underground with the maximum length of 45 meters considering the penetrated lengths of 5 meters, a drainage well should be so designed to arrange horizontal drainage borings at proper depths and locations.

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(f) Arrangements of Borings and Wells
Horizontal drainage borings shall be arranged as per the requirements of Section 3011 – Horizontal Underground Drainage.

Drainage well should be arranges in the following manner:

- The maximum lengths of discharge boring should be less than 80 meters.
- The discharge boring should have enough downward inclination from the well to the outlet for smooth drainage of the water collected by horizontal drainage borings.
- The bottom elevation of a drainage well should be enough deep so that horizontal drainage boring from the drainage well could penetrate the targeted slip surface with enough length of penetration.
- Structure of a drainage well should be so designed that walls of the drainage well can be sustained against earth pressure with the liner plates installed along the walls.

(g) Effect on Slope Stability Analysis
As the effects of the works, reduction of groundwater level in slope stability analysis shall be not more than 5 meters deep.

(h) Drainage System to Lead Drained Water to the Nearest Stream or Drainage Ditches
Along with drainage wells and horizontal drainage borings, drainage system shall be so designed that groundwater flowing from the outlet of the discharge borings from the drainage wells should be duly collected and lead to the nearest stream or drainage ditches which has enough capacity to accommodate additional flow from the drainage wells.

3012.3 Materials

Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Authority's Engineer.

(a) Materials excavated shall be classified as per Specifications for Road and Bridge Works of MoRTH

(b) Liner Plate and Ladder – shall be fabricated as shown on the Drawings and meet the requirements of JIS G 3101 SS 330 or an equivalent standard. Liner Plates and Ladders shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(c) Bolts and Nuts (other than those required for the fencing system) shall be fabricated as

shown on the Drawings and meet the requirements of JIS B 1180 and JIS B 1181 or an equivalent standard. Bolts and Nuts shall have galvanized coatings conforming to JIS H 8641 HDZ 35 or an equivalent standard.

(d) Reinforcing Rings - shall be fabricated as shown on the Drawings and meet the requirements of JIS G 3101 SS 400 or an equivalent standard. Reinforcing Rings shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(e) Perforated Pipes –shall meet the requirements of Section 3011 – Horizontal Underground Drainage.

(f) Protective Pipes – shall meet the requirements of Section 3011 - Horizontal Underground Drainage

(g) Non-perforated steel pipe – shall be steel pipes meeting the requirement in the following table and conforming to JIS G 3452 or an equivalent standard. These pipes shall be connected by screw joints conforming to JIS B 2302 or an equivalent standard.

Type	External diameter	Tolerance of external diameter	Thickness	Tolerance of thickness
SGP 90A	101.6 mm	±0.8mm	4.2mm	-12.5%

(h) Fencing System - shall be fabricated as shown on the Drawings and the sub-components of the fencing system shall meet the following requirements

(i) Struts and furring strips shall meet the requirements of JIS G 3444 or an equivalent standard.

(ii) Wire Net shall meet the requirements of JIS G 3552 or an equivalent standard and conform to the requirements of the Table below:

Diameter of Wire (mm)	Mesh Size (mm)	Width (m)	Length (m)	Unit Weight (kg/m^2)	Zinc Coating (g/m^2)	Tensile Strength (N/mm^2)
2.0	50X50	1.0	10.0	1.0	≥ 23	290 – 540

(iii) Brackets shall meet the requirements of JIS G 3101 or an equivalent standard.

(iv) Bolts and Nuts shall meet the requirements of JIS B 1188 and JIS B 1181 or an equivalent standards.

All the metal sub-components of the fence system shall have galvanized coating conforming to JIS H 8641

(i) Well Cover - shall be fabricated using checkered steel plates and as shown on the Drawings and meet the requirements of JIS G 3193 or an equivalent standard. Liner Plates and Ladders shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(j) Concrete
Concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH.

(k) Reinforcing Bars
Reinforcing bars shall conform to BS4449 Grade 460 or equivalent.

(l) Formwork
Formwork shall conform to the Specifications for Road and Bridge Works of MoRTH.

3012.4 Construction Requirements

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings to the Authority's Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Authority's Engineer has the authority to relax or do away with this requirement, if he chooses to do so, on a written request made by the Contractor.
The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Manufacture of Liner Plates, Reinforcement Rings and Factory Inspection

(i) The Contractor shall not proceed with any purchase of or fabrication of materials until the relevant shop drawings have been approved by the Authority's Engineer. Prior to the use of any materials, the Contractor shall submit for the Authority's Engineer's approval two copies of the Manufacturer's certificate for materials, which shall include the names and locations of mills, results of analysis of chemical and physical properties, and shall be properly correlated to the various grades and delivery batches of materials to be used in fabrication.

(ii) Prior to fabrication, the Contractor shall submit the quality control plan and Quality Assurance Plan for the fabrication workshops for the Authority's Engineer's approval.

(iii) The Liner Plates and the Reinforcement Rings shall be fabricated to the dimensions shown in the Drawings.

(iv) During the manufacturing of the Liner Plates and the Reinforcement Rings, the Authority's Engineer shall have the authority to inspect, examine and test materials, workmanship and performance of any part of the works at the supplier's or the fabricator's facilities.

(v) The Contractor shall provide all facilities for the inspection of material and workmanship at these facilities and provide opportunities to the inspection team to visit the locations where the manufactured items are installed and are

functioning satisfactorily. The Authority's Engineer, his representatives and his delegates shall be allowed free access to all parts of the premises, during all reasonable times, upon request.

(vi) The inspectors shall be appointed by the Authority's Engineer, but all expenses for inspection including fees, cost of return air-travel from the country of the inspectors to the country where the above facilities are located, other transport costs, accommodation costs, and per diem shall be paid by the Contractor and shall be deemed to include in the pay items for the Drainage Well.

(vii) Inspection shall be carried out by a team of 03 inspectors or the Authority's Engineer and 02 inspectors. Each trip of inspection shall typically last for 12 days at the above facilities.

(viii) The inspectors shall take instructions from the Authority's Engineer and his representatives and shall submit inspection reports to the Authority's Engineer. The inspectors shall certify that all works were performed by the Contractor or suppliers up to the stage of fabrication at the factory have been carried out in accordance with this Section and the approved shop drawings.

(ix) Relevant authority of the Authority's Engineer will be delegated to the inspectors for the purpose of quality control and testing. The Contractor shall be informed of the authority so delegated, while the inspectors are appointed.

(x) The inspector's certificate shall not relieve the Contractor of any of his obligations under the Contract

(c) Digging the well, installing the liner plates and ladders and installing the reinforcement rings

The location where the well is shown on the Drawing shall be cleared as per the Specifications for Road and Bridge Works of MoRTH and the ground prepared by removing the earth and boulders up to the level shown in the Drawings, before the commencement of the excavation.
The methodology of excavation shall be selected considering the soil properties, design requirements, ground conditions and the site conditions. Motorized machinery shall be used for excavation, in order to overcome the oxygen deficiency. If rock or boulders are encountered in the well, chemical expansion method or any other suitable method at the Contractor's choice and acceptable to the Authority's Engineer shall be used for blasting them.

Employing chemical expansion method of rock / boulder excavation shall have 14 days prior approval of the Authority's Engineer. The method statement for chemical expansion shall be accompanied by manufacturer's certificates and catalogues of the chemical to be used.
To avoid the collapse of the well walls, before the liner plate is installed, the unlined depth of excavation shall not exceed 0.5 m at any time.

The water entering the dug pit shall be removed by suitable methods, in order to facilitate trouble free excavation. The Contractor shall not be paid for dealing with water and it shall be deemed to be included in the rates.

The installation of the liner plates shall progress along with the excavation. The reinforcing rings shall be installed at required elevations as shown in the Drawing or as directed by the Authority's Engineer. In order to install the reinforcing rings, additional excavation may be necessary at the locations where the nuts and bolts securing the reinforcement rings with the liner plates are located. The cost connected to such additional excavation shall not be paid to the Contractor and deemed to be included in the rates for excavation. The provision and installation of reinforcing rings shall not be paid separately and deemed to be included in the rate for liner plates.

The ladder shall be installed after completion of excavation, horizontal underground drainage installation, discharge drainage installation and removal of all scaffolding and temporary works except those which are needed for the installation of ladders.
The surplus material resulting from the excavation shall be disposed to the dump areas located by the Contractor and approved by the Authority's Engineer. The Contractor shall not be separately paid for this and the rates for excavation shall be deemed to include the cost of this.

Samples of material encountered during excavation shall be collected at every 0.5m from the top of the well and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label shall include information such as identification number, location within the well where the sample was collected etc. Collection of samples and submission of excavation reports with photographs shall not be measured nor paid for and deemed to be included in the rate for excavation.

In case the landslide clay, a type of clay derived from landslide movement and found at slip surfaces, is encountered, the Contractor shall report it to the Authority's Engineer and secure undisturbed sample of the landslide clay for the use of laboratory testing such as direct sheer test or tri-axial compressive test. The manner to obtain and storage undisturbed sample of landslide clay shall be proposed by the Contractor and approved by the Authority's Engineer. Collection of undisturbed samples shall not be measured nor paid for and deemed to be included in the rate for excavation.

Immediately after the excavation is completed for each 0.5 m and before the installation of the liner plate, the Contractor shall make records of excavated walls by taking photographs and sketches. Based on such information, within 14 days of the completion of the excavation, the Contractor shall submit an excavation report to the Authority's Engineer, accurately describing the boundaries of the geological formations, seepage of water or volume of groundwater pumped out from the well. The format of the excavation report shall have the prior approval of the Authority's Engineer.

(d) Construction of Well Platform

The Well Platform shall be constructed as per the details given in the Drawings or as instructed by the Authority's Engineer.

In order to construct the Well Platform some excavation may be necessary. But this

excavation shall not be paid for. The liner of the drainage well shall be used as the formwork for the curved internal surface of the Well Platform. The liner plates shall be installed vertically so as to ensure the differences of elevation at four locations which are equally spaced around the perimeter of the top of the liner plates are less than 5 mm.

The concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH (Standard Type, Gradee M20, and maximum size of aggregate 37,5mm).

Backfilling shall be done around the platform as per the Specifications for Road and Bridge Works of MoRTH.

The Well Platform shall have a name plate giving the relevant details for the easy identification of the drainage well.

(e) Drilling of Discharge Boring

The contractor shall use appropriate equipment for drilling the holes, taking into consideration the factors such as soil properties, ground conditions, design requirements, site conditions etc. The design requirements include, among other things, the diameter of the non-perforated steel pipe. The bore hole shall be protected by temporary casings of appropriate diameter till the non-perforated steel pipes are in place. No payment shall be made for provision and insertion of these temporary casings and the rate for Non-perforated Steel Pipes shall be deemed to compensate for this.

Samples of material encountered shall be collected at every 1.0m of the drill hole and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label shall include information such as identification number, location within the drill hole where the sample was collected etc.

Within 14 days of the completion of the drilling of each hole, the contractor shall submit a drilling report to the Authority's Engineer, accurately describing the boundaries of the geological formations, location of incoming groundwater in the drill-hole etc. The format of the drilling report shall have the prior approval of the Authority's Engineer.

The Contractor shall pull out the drilling rods, in the presence of the Authority's Engineer and the length of the drilling rods shall be measured by the Engineer and the Contractor, which shall form the basis of deciding the depth of the drill-hole.

The drill holes shall be within the following tolerances:

Category	Tolerance (mm)
Length	Design length or more
Displacement	100.0
Direction	±2.5 degrees in any direction

Connecting the outlet from the drainage well to the non-perforated steel pipe, collection of samples and submission of drilling reports shall not be measured nor paid for and deemed to be included in the rate for drilling

(f) Installing Non-perforated Steel Pipes

Before the commencement of the installation, the Contractor shall obtain permission from the Authority's Engineer to do so.

The non-perforated steel pipes shall be installed into the whole length of the drill hole. These pipes shall be inserted through the mouth of the drill hole which is away from the drilling machine and the temporary casing shall be withdrawn from the mouth of the drill hole which is closer to the drilling machine. If the pipes are to be connected, the connections shall be sturdy so that there will be no loosening of connections. The connections shall be of screw type which shall meet the requirements in accordance with JIS B 2302 or an equivalent standard.

(g) Horizontal drilling and Installing the perforated pipes

Drilling for horizontal perforated pipes and installation of Perforated pipes shall be in accordance with Section 3011 of these Specifications.

Connecting the inlet from the perforated pipes with the drainage well shall not be measured nor paid for and deemed to be included in the rate for drilling

(h) Base Concrete

The Base Concrete for the Well shall be constructed as per the details given in the Drawings or as instructed by the Authority's Engineer.

The concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH (Standard Type, Gradee M20, and maximum size of aggregate 37,5mm).

(i) Fencing System

The Fencing System for the Well shall be constructed as per the details given in the Drawings or as instructed by the Authority's Engineer.

The fence shall have a name plate giving the relevant details for the easy identification of the drainage well.

(j) Cover for the Well

The Cover for the Well shall be constructed as per the details given in the Drawings or as instructed by the Authority's Engineer. The door plate of the cover shall have a robust and weather proof locking device.

(k) Gabions

Gabions shall be constructed as per the Drawings or as directed by the Authority's Engineer and Specifications for Road and Bridge Works of MoRTH.

(l) Inspection and Maintenance Equipment

The Contractor shall supply the following inspection and maintenance equipment and tools and accessories to the Employer before making a request for the Taking – Over Certificate.

The User's Manual, Repair Manual and Maintenance Manual shall be in English.

Warning information and any other information fixed to the equipment shall be in English.

Requirement of Inspection and Maintenance Equipment and Tools

(The brand names and model numbers given are only indicative. Equivalent brands and models are also acceptable)

Item	Manufacturer	Model	Basic Requirements	No of Units
High Pressure water sprayers used for construction	SHINSO Co. Ltd.	SJD-2044SAS	Pressure 20MPa, Water Qty 44 lit/min, Diesel Engine 27Ps	01
High Pressure rubber house (Machine hose bottom)	SHINSO Co. Ltd.	SJD-2044SAS	Withstand Pressure 20.5 MPa, Φ12x30cm ½ inch installing coupler	01
Foot valve	SHINSO Co. Ltd.	FV-230	Pressure 20 MPa, Water Qty. 30 lit/min	01
High pressure jet hose (Discharge Boring)	SHINSO Co. Ltd.	SKN-6H	Withstand Pressure 20 MPa, Φ9.6x85cm ⅜ inch installing coupler	01
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR9-F34FRR	⅜ inch one nozzle on the front side, two nozzles on the backside	01
Hairpin rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR9-A34-N	⅜ inch, two nozzles on the backside	01
High pressure jet hose (Horizontal drainage Boring)	SHINSO Co. Ltd.	SKN-4H	Withstand Pressure 30 MPa, Φ6.4x85cm ⅜ inch installing coupler	01
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR6-FA25RR-N	With front side nozzle	01
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR6-A25RR-N	Only backside nozzle	01
Water tank	SUIKO CO., LTD.	SLT-2000	1550*1550*1200 Volume	01

			2000 L	
Feed pump (Petrol engine)	MARUYAMA MFG. CO., INC.	MS415EA	Output 4.7Ps, Pressure 5MPa. Suction force 30L/min	01
Water supply hose	MARUYAMA MFG. CO., INC.	AR50	φ8.5×50m	01

Requirement of Inspection and Maintenance Accessories

(The brand names and model numbers given are only indicative. Equivalent brands and models are also acceptable)

Item	Manufacturer	Model	Basic Requirements	No of Units
Diesel generator	DENYO CO.,Ltd.	DCA-25LSKE	Soundproof type, Output minimum 20kW	01
Submersible pump (High lift type)	TSURUMI PUMP CO.,LTD.	KTV2-37H	Output 3.7kW, Pipe size 50mm, Three-phase 200 or 440V, Total pump head 30m, 0.2m3/min	01
Air Blower	YAMABIKO CORPORATION	EPV300A	Pipe size 300mm, Power voltage 100 or 230 V, Output 0.55kW, Maximum wind force 60/70m3/min	01
Floodlight	NICHIDO IND. CO., LTD.	ATL-1410-5000K	Emission color 5000K(Day time), Rated luminous flux 1470Lm, Light source LED 14W, Power voltage 230 kV	01
Multi gas detector	NEW COSMOS ELECTRIC CO., LTD.	XP-302M-A-4 (with reel)	4 building block type ・Flammable gas (Methane gas, CH4) 0～100% LEL ・Oxygen (O2) 0～25.0vol% ・Hydrogen sulfide (H2S) 0～30.0ppm ・Carbon monoxide (OC) 0～150ppm ・Sensors shall have a code of 8 meters long or more.	01
Handy light (LED Zoom)	OHM ELECTRIC CO., LTD.	LED-PY102ZK	LED type, Water proof, Switch 3 steps brightness, Dry-cell battery	01
Pipe inspection camera and light	REX CO., LTD	KDM-200SLM	Pipe type φ40～250mm, Cable length 60m	01

3012.5 Record

(a) Record
The necessary data for maintenance and management shall be recorded and preserved.

Note

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3012 DRAINAGE WELL (For Item Rate)

3012.1 Description

This work shall consist of constructing wells and providing associated facilities in order to collect water from the land slide / slope failure prone areas and safely discharge the water so collected into natural or man-made canals or drains.

Well excavation shall consist of all the required excavation within the well. The work shall include the removal, stockpiling, handling hauling and proper utilization in the works or disposal to spoil tips, located by the Contractor and approved by the Engineer, of all excess excavated material.

3012.2 Materials

Materials shall comply with the IRC's prescribed standards, Indian Standards, Japanese Industrial Standards, other international standards, or equivalent, and have approvals from the Engineer.

(a) Materials excavated shall be classified as per Specifications for Road and Bridge Works of MoRTH

(b) Liner Plate and Ladder – shall be fabricated as shown on the Drawings and meet the requirements of JIS G 3101 SS 330 or an equivalent standard. Liner Plates and Ladders shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(c) Bolts and Nuts (other than those required for the fencing system) shall be fabricated as shown on the Drawings and meet the requirements of JIS B 1180 and JIS B 1181 or an equivalent standard. Bolts and Nuts shall have galvanized coatings conforming to JIS H 8641 HDZ 35 or an equivalent standard.

(d) Reinforcing Rings - shall be fabricated as shown on the Drawings and meet the requirements of JIS G 3101 SS 400 or an equivalent standard. Reinforcing Rings shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(e) Perforated Pipes –shall meet the requirements of Section 3011 – Horizontal Underground Drainage.

(f) Protective Pipes – shall meet the requirements of Section 3011 - Horizontal Underground Drainage

(g) Non-perforated steel pipe – shall be steel pipes meeting the requirement in the following table and conforming to JIS G 3452 or an equivalent standard. These pipes shall be connected by screw joints conforming to JIS B 2302 or an equivalent standard.

Type	External diameter	Tolerance of external diameter	Thickness	Tolerance of thickness
SGP 90A	101.6 mm	±0.8mm	4.2mm	-12.5%

(h) Fencing System - shall be fabricated as shown on the Drawings and the sub-components of the fencing system shall meet the following requirements

(i) Struts and furring strips shall meet the requirements of JIS G 3444 or an equivalent standard.

(ii) Wire Net shall meet the requirements of JIS G 3552 or an equivalent standard and conform to the requirements of the Table below:

Diameter of Wire (mm)	Mesh Size (mm)	Width (m)	Length (m)	Unit Weight (kg/m^2)	Zinc Coating (g/m^2)	Tensile Strength (N/mm^2)
2.0	50X50	1.0	10.0	1.0	≥ 23	290 – 540

(iii) Brackets shall meet the requirements of JIS G 3101 or an equivalent standard.

(iv) Bolts and Nuts shall meet the requirements of JIS B 1188 and JIS B 1181 or an equivalent standards.

All the metal sub-components of the fence system shall have galvanized coating conforming to JIS H 8641

(i) Well Cover - shall be fabricated using checkered steel plates and as shown on the Drawings and meet the requirements of JIS G 3193 or an equivalent standard. Liner Plates and Ladders shall have galvanized coatings conforming to JIS H 8641 HDZ 45 or an equivalent standard.

(j) Concrete
Concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH.

(k) Reinforcing Bars
Reinforcing bars shall conform to BS4449 Grade 460 or equivalent.

(l) Formwork
Formwork shall conform to the Specifications for Road and Bridge Works of MoRTH.

3012.3 Construction Requirements

(a) Method Statements, Working Drawings / Shop Drawings
The Contractor shall submit the Methods Statements, Working Drawings / Shop Drawings
to the Engineer, for his approval, at least 28 days prior to the commencement of the relevant activity. The Engineer has the authority to relax or do away with this requirement,
if he chooses to do so, on a written request made by the Contractor.
The above documents shall clearly spell out the method the Contractor proposes to carry out the construction / fabrication of the Permanent Works and Temporary Works, carrying out the quality control tests and their frequency. They shall also further give details of safety measures and the equipment and personnel to be deployed to carry out the works. No payment will be made for preparing the above documents.

(b) Manufacture of Liner Plates, Reinforcement Rings and Factory Inspection

(i) The Contractor shall not proceed with any purchase of or fabrication of materials

until the relevant shop drawings have been approved by the Engineer. Prior to the use of any materials, the Contractor shall submit for the Engineer's approval two copies of the Manufacturer's certificate for materials, which shall include the names and locations of mills, results of analysis of chemical and physical properties, and shall be properly correlated to the various grades and delivery batches of materials to be used in fabrication.

(ii) Prior to fabrication, the Contractor shall submit the quality control plan and Quality Assurance Plan for the fabrication workshops for the Engineer's approval.

(iii) The Liner Plates and the Reinforcement Rings shall be fabricated to the dimensions shown in the Drawings.

(iv) During the manufacturing of the Liner Plates and the Reinforcement Rings, the Engineer shall have the authority to inspect, examine and test materials, workmanship and performance of any part of the works at the supplier's or the fabricator's facilities.

(v) The Contractor shall provide all facilities for the inspection of material and workmanship at these facilities and provide opportunities to the inspection team to visit the locations where the manufactured items are installed and are functioning satisfactorily. The Engineer, his representatives and his delegates shall be allowed free access to all parts of the premises, during all reasonable times, upon request.

(vi) The inspectors shall be appointed by the Engineer, but all expenses for inspection including fees, cost of return air-travel from the country of the inspectors to the country where the above facilities are located, other transport costs, accommodation costs, and per diem shall be paid by the Contractor and shall be deemed to include in the pay items for the Drainage Well.

(vii) Inspection shall be carried out by a team of 03 inspectors or the Engineer and 02 inspectors. Each trip of inspection shall typically last for 12 days at the above facilities.

(viii) The inspectors shall take instructions from the Engineer and his representatives and shall submit inspection reports to the Engineer. The inspectors shall certify that all works were performed by the Contractor or suppliers up to the stage of fabrication at the factory have been carried out in accordance with this Section and the approved shop drawings.

(ix) Relevant authority of the Engineer will be delegated to the inspectors for the purpose of quality control and testing. The Contractor shall be informed of the authority so delegated, while the inspectors are appointed.

(x) The inspector's certificate shall not relieve the Contractor of any of his obligations under the Contract

(c) Digging the well, installing the liner plates and ladders and installing the reinforcement rings

The location where the well is shown on the Drawing shall be cleared as per the Specifications for Road and Bridge Works of MoRTH and the ground prepared by removing the earth and boulders up to the level shown in the Drawings, before the commencement of the excavation.
The methodology of excavation shall be selected considering the soil properties, design requirements, ground conditions and the site conditions. Motorized machinery shall be used for excavation, in order to overcome the oxygen deficiency. If rock or boulders are encountered in the well, chemical expansion method or any other suitable method at the Contractor's choice and acceptable to the Engineer shall be used for blasting them.

Employing chemical expansion method of rock / boulder excavation shall have 14 days prior approval of the Engineer. The method statement for chemical expansion shall be accompanied by manufacturer's certificates and catalogues of the chemical to be used. To avoid the collapse of the well walls, before the liner plate is installed, the unlined depth of excavation shall not exceed 0.5 m at any time.

The water entering the dug pit shall be removed by suitable methods, in order to facilitate trouble free excavation. The Contractor shall not be paid for dealing with water and it shall be deemed to be included in the rates.

The installation of the liner plates shall progress along with the excavation. The reinforcing rings shall be installed at required elevations as shown in the Drawing or as directed by the Engineer. In order to install the reinforcing rings, additional excavation may be necessary at the locations where the nuts and bolts securing the reinforcement rings with the liner plates are located. The cost connected to such additional excavation shall not be paid to the Contractor and deemed to be included in the rates for excavation. The provision and installation of reinforcing rings shall not be paid separately and deemed to be included in the rate for liner plates.

The ladder shall be installed after completion of excavation, horizontal underground drainage installation, discharge drainage installation and removal of all scaffolding and temporary works except those which are needed for the installation of ladders.
The surplus material resulting from the excavation shall be disposed to the dump areas located by the Contractor and approved by the Engineer. The Contractor shall not be separately paid for this and the rates for excavation shall be deemed to include the cost of this.

Samples of material encountered during excavation shall be collected at every 0.5m from the top of the well and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label shall include information such as identification number, location within the well where the sample was collected etc. Collection of samples and submission of excavation reports with photographs shall not be measured nor paid for and deemed to be included in the rate for excavation.

In case the landslide clay, a type of clay derived from landslide movement and found at slip surfaces, is encountered, the Contractor shall report it to the Engineer and secure undisturbed sample of the landslide clay for the use of laboratory testing such as direct

sheer test or tri-axial compressive test. The manner to obtain and storage undisturbed sample of landslide clay shall be proposed by the Contractor and approved by the Engineer. Collection of undisturbed samples shall not be measured nor paid for and deemed to be included in the rate for excavation.

Immediately after the excavation is completed for each 0.5 m and before the installation of the liner plate, the Contractor shall make records of excavated walls by taking photographs and sketches. Based on such information, within 14 days of the completion of the excavation, the Contractor shall submit an excavation report to the Engineer, accurately describing the boundaries of the geological formations, seepage of water or volume of groundwater pumped out from the well. The format of the excavation report shall have the prior approval of the Engineer.

(d) Construction of Well Platform

The Well Platform shall be constructed as per the details given in the Drawings or as instructed by the Engineer.

In order to construct the Well Platform some excavation may be necessary. But this excavation shall not be paid for. The liner of the drainage well shall be used as the formwork for the curved internal surface of the Well Platform. The liner plates shall be installed vertically so as to ensure the differences of elevation at four locations which are equally spaced around the perimeter of the top of the liner plates are less than 5 mm.

The concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH (Standard Type, Gradee M20, and maximum size of aggregate 37,5mm).

Backfilling shall be done around the platform as per the Specifications for Road and Bridge Works of MoRTH.

The Well Platform shall have a name plate giving the relevant details for the easy identification of the drainage well.

(e) Drilling of Discharge Boring

The contractor shall use appropriate equipment for drilling the holes, taking into consideration the factors such as soil properties, ground conditions, design requirements, site conditions etc. The design requirements include, among other things, the diameter of the non-perforated steel pipe. The bore hole shall be protected by temporary casings of appropriate diameter till the non-perforated steel pipes are in place. No payment shall be made for provision and insertion of these temporary casings and the rate for Non-perforated Steel Pipes shall be deemed to compensate for this.

Samples of material encountered shall be collected at every 1.0m of the drill hole and at places the type of material changes. These samples shall be neatly packed in transparent polythene and appropriately labelled for easy identification and interpretation. The label

shall include information such as identification number, location within the drill hole where the sample was collected etc.

Within 14 days of the completion of the drilling of each hole, the contractor shall submit a drilling report to the Engineer, accurately describing the boundaries of the geological formations, location of incoming groundwater in the drill-hole etc. The format of the drilling report shall have the prior approval of the Engineer.

The Contractor shall pull out the drilling rods, in the presence of the Engineer and the length of the drilling rods shall be measured by the Engineer and the Contractor, which shall form the basis of deciding the depth of the drill-hole.

The drill holes shall be within the following tolerances:

Category	Tolerance (mm)
Length	Design length or more
Displacement	100.0
Direction	±2.5 degrees in any direction

Connecting the outlet from the drainage well to the non-perforated steel pipe, collection of samples and submission of drilling reports shall not be measured nor paid for and deemed to be included in the rate for drilling

(f) Installing Non-perforated Steel Pipes

Before the commencement of the installation, the Contractor shall obtain permission from the Engineer to do so.

The non-perforated steel pipes shall be installed into the whole length of the drill hole. These pipes shall be inserted through the mouth of the drill hole which is away from the drilling machine and the temporary casing shall be withdrawn from the mouth of the drill hole which is closer to the drilling machine. If the pipes are to be connected, the connections shall be sturdy so that there will be no loosening of connections. The connections shall be of screw type which shall meet the requirements in accordance with JIS B 2302 or an equivalent standard.

(g) Horizontal drilling and Installing the perforated pipes

Drilling for horizontal perforated pipes and installation of Perforated pipes shall be in accordance with Section 3011 of these Specifications.

Connecting the inlet from the perforated pipes with the drainage well shall not be measured nor paid for and deemed to be included in the rate for drilling

(h) Base Concrete

The Base Concrete for the Well shall be constructed as per the details given in the Drawings or as instructed by the Engineer.

The concrete shall conform to Section 1700 of the Specifications for Road and Bridge Works of MoRTH (Standard Type, Gradee M20, and maximum size of aggregate 37,5mm).

(i) Fencing System

The Fencing System for the Well shall be constructed as per the details given in the Drawings or as instructed by the Engineer.

The fence shall have a name plate giving the relevant details for the easy identification of the drainage well.

(j) Cover for the Well

The Cover for the Well shall be constructed as per the details given in the Drawings or as instructed by the Engineer. The door plate of the cover shall have a robust and weather proof locking device.

(k) Gabions

Gabions shall be constructed as per the Drawings or as directed by the Engineer and Specifications for Road and Bridge Works of MoRTH.

(l) Inspection and Maintenance Equipment

The Contractor shall supply the following inspection and maintenance equipment and tools and accessories to the Employer before making a request for the Taking – Over Certificate.

The User's Manual, Repair Manual and Maintenance Manual shall be in English. Warning information and any other information fixed to the equipment shall be in English.

Requirement of Inspection and Maintenance Equipment and Tools

(The brand names and model numbers given are only indicative. Equivalent brands and models are also acceptable)

Item	Manufacturer	Model	Basic Requirements	No of Units
High Pressure water sprayers used for construction	SHINSO Co. Ltd.	SJD-2044SAS	Pressure 20MPa, Water Qty 44 lit/min, Diesel Engine 27Ps	01
High Pressure rubber house (Machine hose bottom)	SHINSO Co. Ltd.	SJD-2044SAS	Withstand Pressure 20.5 MPa, Φ12x30cm ½ inch installing coupler	01
Foot valve	SHINSO Co. Ltd.	FV-230	Pressure 20 MPa, Water Qty. 30 lit/min	01
High pressure jet	SHINSO Co. Ltd.	SKN-6H	Withstand Pressure 20 MPa.	01

hose (Discharge Boring)			Φ9.6x85cm ⅜ inch installing coupler	
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR9-F34FRR	⅜ inch one nozzle on the front side, two nozzles on the backside	01
Hairpin rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR9-A34-N	⅜ inch, two nozzles on the backside	01
High pressure jet hose (Horizontal drainage Boring)	SHINSO Co. Ltd.	SKN-4H	Withstand Pressure 30 MPa, Φ6.4x85cm ⅜ inch installing coupler	01
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR6-FA25RR-N	With front side nozzle	01
High pressure rotor nozzle (Discharge boring)	SHINSO Co. Ltd.	HR6-A25RR-N	Only backside nozzle	01
Water tank	SUIKO CO., LTD.	SLT-2000	1550*1550*1200 Volume 2000 L	01
Feed pump (Petrol engine)	MARUYAMA MFG. CO., INC.	MS415EA	Output 4.7Ps, Pressure 5MPa, Suction force 30L/min	01
Water supply hose	MARUYAMA MFG. CO., INC.	AR50	φ8.5×50m	01

Requirement of Inspection and Maintenance Accessories

(The brand names and model numbers given are only indicative. Equivalent brands and models are also acceptable)

Item	Manufacturer	Model	Basic Requirements	No of Units
Diesel generator	DENYO CO.,Ltd.	DCA-25LSKE	Soundproof type, Output minimum 20kW	01
Submersible pump (High lift type)	TSURUMI PUMP CO.,LTD.	KTV2-37H	Output 3.7kW, Pipe size 50mm, Three-phase 200 or 440V, Total pump head 30m, 0.2m3/min	01
Air Blower	YAMABIKO CORPORATION	EPV300A	Pipe size 300mm, Power voltage 100 or 230 V, Output 0.55kW, Maximum wind force 60/70m3/min	01
Floodlight	NICHIDO IND. CO., LTD.	ATL-1410-5000K	Emission color 5000K(Day time), Rated luminous flux 1470Lm, Light source LED 14W, Power voltage 230 kV	01

Multi gas detector	NEW COSMOS ELECTRIC CO., LTD.	XP-302M-A-4 (with reel)	4 building block type ・Flammable gas (Methane gas, CH4) 0～100% LEL ・Oxygen (O2) 0～25.0vol% ・Hydrogen sulfide (H2S) 0～30.0ppm ・Carbon monoxide (OC) 0～150ppm ・Sensors shall have a code of 8 meters long or more.	01
Handy light (LED Zoom)	OHM ELECTRIC CO., LTD.	LED-PY102ZK	LED type, Water proof, Switch 3 steps brightness, Dry-cell battery	01
Pipe inspection camera and light	REX CO., LTD	KDM-200SLM	Pipe type φ40～250mm, Cable length 60m	01

3012.4 Record

(a) Record
The necessary data for maintenance and management shall be recorded and preserved.

3012.5 Measurement and Payment

(a) Measurement

Clearing and Grubbing

Clearing and grubbing shall be measured as per Specifications for Road and Bridge Works of MoRTH.

Ground Preparation

The Ground Preparation will be measured as a Lump Sum, irrespective of the material encountered during the preparation of the ground for the excavation of well and the associated facilities.

Well Platform

Well Platform shall be measured in numbers as the number of well platforms constructed.

Well Excavation

The quantity of excavation shall be measured in linear meters of the depth of excavation completed. The depth of excavation through various types of soil / boulders/ rock will be based on the excavation report submitted by the Contractor and accepted by the Engineer. Materials encountered in the excavation shall be classified as per specifications for Road and Bridge Works of MoRTH," but Hard Rock and Boulders shall be classified as Rock for the purpose of this Section, soft rock as soft rock and the other classes will be classified as soil for the purpose of this Section. When chemical expansion method is used to excavate boulders/rock, the cost of using such method shall

be included in the rates for boulder / rock excavation.

The additional excavation needed for base concrete and connecting the reinforcement rings to the liner plates shall not be measured separately and deemed to be included in the well excavation measured in linear meters.

Liner Plates and Ladders
The liner plate shall be measured in linear meters of circular sections, including the ladders within the measured length. The measurement shall be taken along the axis of the circular sections installed.

Reinforcement Rings
Reinforcement Rings shall be measured in numbers as the number of reinforcement rings provided and installed in place.

Base Concrete
Base Concrete shall be measured in numbers as the number of well bases constructed.

Drilling for Discharge of Water
The depth of discharge drilling completed for installation of non-perforated steel pipe is measured in linear meters by the length of the drill rods used for such drilling. The length of discharge drilling through various types of soil / boulders/ rock will be based on the drilling report submitted by the Contractor and accepted by the Engineer

Non-perforated Steel Pipes
Non-perforated steel pipe shall be measured in linear meters, by the length of pipes installed. The connections shall not be paid separately and deemed to be included in the rates for non-perforated steel pipes.

Horizontal Drilling
Horizontal drilling and installation of perforated pipes shall be measured as per Section 3011.

Well Covers
Well Cover shall be measured in numbers as the number of well covers provided and installed in place.

Fencing System
Fencing System shall be measured in numbers as the number of fencing systems provided and installed in place.

Inspection and Maintenance Equipment
Inspection and Maintenance Equipment shall be measured in sets of equipment, tools and accessories supplied and accepted. Each set shall constitute all the equipment listed.

Gabions
Gabions shall be measured as per Specifications for Road and Bridge Works of MoRTH.

(b) Payment

All prices shall include full compensation for all labour, materials, tools, equipment incidentals etc. necessary to complete the work to these Specifications and in addition shall include the following where applicable :-

1. Setting out
2. Planking and any other precautionary measure taken to ensure the stability of adjacent areas.
3. Removal of excess or unsuitable materials, logs, stumps etc.
4. Safety measures
5. Measures required for working under restricted conditions.
6. Laboratory and field testing
6. Dewatering

Clearing and Grubbing
Clearing and grubbing shall be paid as per Specifications for Road and Bridge Works of MoRTH.

Ground Preparation
The Ground Preparation will be paid as a Lump Sum and all that is necessary to prepare the ground for the work.

Well Platform
This shall be paid at the Contract unit rate per number and shall include the formwork, reinforcement and concrete and back filling according to the Drawings and these Specifications and to the satisfaction of the Engineer

Well Excavation
The quantities of excavation measured as specified above will be paid for at the Contract unit rates per linear meter for the various types as detailed below. Such rates shall include excavation, preparation of excavation report, removal, stockpiling, multiple handling, haulage to other areas of the Site for re-use and satisfactory disposal of all excavation, to designated spoil tips or spoil tips located by the Contractor and approved by the Engineer, for shaping, dressing and completion of all surfaces..

Liner Plates and Ladders
These shall be paid at the Contract unit rate per linear meter and shall include the provision of all the materials, the fabrication, transport to site and installation according to the Drawings and these Specifications and to the satisfaction of the Engineer. The liner plates, reinforcement rings and ladders shall be treated as a composite item for payment.

Reinforcement Rings
This shall be paid at the Contract unit rate per number and shall include the provision of

all the materials, the fabrication, transport to site and installation according to the Drawings and these Specifications and to the satisfaction of the Engineer.

Base Concrete

This shall be paid at the Contract unit rate per number and shall include the formwork, reinforcement and concrete and back filling according to the Drawings and these Specifications and to the satisfaction of the Engineer.

Drilling for Discharge of Water

The accepted quantities of the drilling shall be paid for at the Contract unit price per linear meter of the drill-hole.

Non-perforated Steel Pipes

The accepted quantities of pipe shall be paid for at the Contract unit price per linear meter of the type and size of pipe as specified, complete in place

Horizontal Drilling

Horizontal drilling and installation of perforated pipes shall be paid as per Section 3011.

Well Covers

This shall be paid at the Contract unit rate per number and shall include the provision of all the materials, the fabrication, transport to site and installation according to the Drawings and these Specifications and to the satisfaction of the Engineer.

Fencing System

This shall be paid at the Contract unit rate per number and shall include the provision of all the materials, the fabrication, transport to site and installation according to the Drawings and these Specifications and to the satisfaction of the Engineer.

Inspection and Maintenance Equipment

These shall be paid at the Contract unit rate per set and shall include the supply of the equipment as per specifications and acceptance by the Engineer. The cost of, demonstration the working of the equipment, training of personnel of the Employer on the use of these equipment and the User's Manual, Maintenance Manual and Repair Manual also shall be included in the rate.

Gabions

Gabions shall be paid as per Specifications for Road and Bridge Works of MoRTH.

Pay Items and Pay Units shall be as follows:

Pay Item	Description	Pay Unit
3012(1)	Ground preparation	LS
3012(2)	Well Platform	Number
3012(3)	Well excavation, soil suitable for fill / unsuitable soil / soft rock	Linear meter

3012(4)	Not used	Linear meter
3012(5)	Well excavation, boulders or hard rock	Linear meter
3012(6)	Liner Plates and Ladders	Linear meter
3012(7)	Reinforcement Rings	Number
3012(8)	Base Concrete	Number
3012(9)	Drilling in suitable or unsuitable soils or soft rock for discharge of water (state the Type of pipe to be inserted)	Linear meter
3012(10)	Drilling in boulders or hard rock for discharge of water (state the Type of pipe to be inserted)	Linear meter
3012(11)	Non-perforated steel pipes (state Type)	Linear meter
3012(12)	Well Cover	Number
3012(13)	Fencing System	Number
3012(14)	Inspection and Maintenance Equipment	Set

Note

DRAFT

Appendix VII

Sample Method Statements:

Cast-in-situ Concrete Crib Works

Horizontal Drainage Boring

METHOD STEATEMENT

施工計画

施工管理

仮設計画

CAST-IN-SITUCONCRETE CRIB WORKS

現場打ちコンクリート枠工

	QA/QC 品質管理
②	
	ete design mixed proportion コンクリー 算書
	test コンクリートのスランプ試験
	ial compressive strength test コンクリ 縮強度試験
	olume test for ready-mixed concrete コ トの空気量測定
	ial 使用材料
	(al 使用材料
	ete design mixed proportion コンクリー 算書
	test コンクリートのスランプ試験
	al compressive strength test コンクリ

	縮強度試験
	olume test for ready-mixed concrete コトの空気量測定

Work Plan 施工計画

① Preparation slope works 準備工

1. Prepare a work rope (master main rope). It is desirable that the rope is about φ18mm and has a tensile strength of 20kN or more.
   Install a work rope above the slope to be work area. The work rope is tied to two standing trees. (If there is no standing tree, two anchors with a diameter of 19mm or more)
   The length of work rope shall be up to the bottom of the slope.
   ロープ作業用のロープ（親綱）を準備する。ロープはφ18 mm程度で引張強度が20kn以上のものが望ましい。
   作業用ロープを工事対象法面の上部へ設置する。作業用ロープは2本の立ち木（立木が無い場合はφ19 mmのアンカー2本）へ結束する。
   作業用ロープの長さは斜面の法尻までとする。

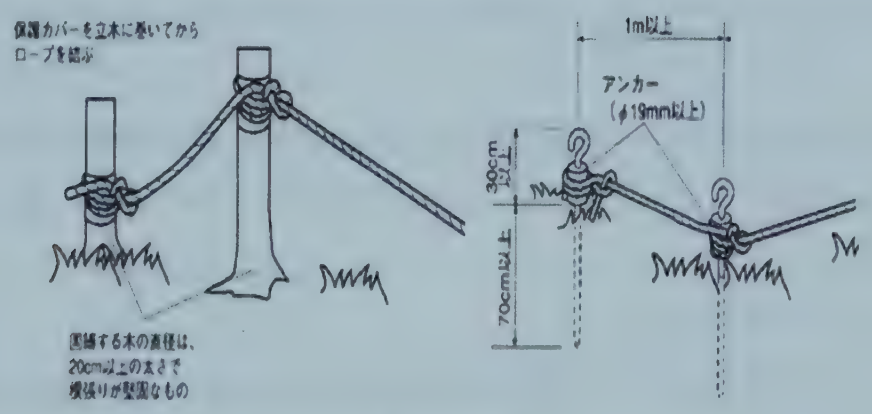


Figure example : Work rope installation

2. During the construction period, an inspection passage will be installed up to the upper part of the slope for daily check of slope abnormalities.
   It is desirable to have handrails along the inspection passage to prevent falls from slopes during slope check and other work.
   工事期間中に斜面の異常を日々点検するための点検用通路を斜面上部まで設置する。
   点検中、移動中に斜面からの転落を防ぐために手すりがあることが望ましい。
3. The person in charge will check the slope. The daily check contents include the presence or absence of spring water, traces of water collection points during rainfall, and the presence or absence of cracks in the upper part of the slope.
   If the spring water is generated constantly or from the slope during rainfall, the cause of the spring water will be investigated, and measures and treatments such as drainage work will be implemented before starting the

slope protection work.
斜面の点検を実施する。点検内容は湧水の有無、降雨時の集水箇所履歴、斜面上部個所へのクラックの有無を点検する。
湧水が恒常的または降雨時に斜面から発生している場合は発生原因を調査し排水工等の対策、処理を実施した後、斜面保護工を開始する。

② Excavation, Trimming and Clearing 基面処理工

1. Excavate the slope according to the planned cross section.
   計画された断面通りに斜面を掘削する。
2. Remove pumice, floating sand, grass, rhizomes, etc. on the slope surface.
   斜面表面の浮石、浮土砂、草、根茎等を除去する。
3. Trim the slope smoothly.
   When assembling the formwork, the formwork material used is generally linear at the lower end peripheral portion, so that a gap due to non-landing occurs between the assembled formwork and the slope. Therefore, it is necessary to level it flat so that there are no gaps. In addition, this gap must be filled in advance by some means. Otherwise, the ready-mixed concrete casting in the formwork will flow out from the gap onto the slope outside the formwork.
   斜面を平坦に整形する。
   型枠組立する際、使用する型枠材料は一般にその下端周縁部が直線的であるため、組み立てた型枠と法面との間に不陸による隙間が生じる。
   そのために、隙間ができないように平坦に均すことが必要である。
   また、この隙き間を何等かの手段によって予め埋めておかなければならない。そうしないと、型枠内に吹付けて打設した生コンクリートが隙き間から型枠外の法面上に流れ出してしまう。

③ Foundation concrete　基礎コンクリート

1. Dig up to the planned position, reference height and width
   計画された位置、基準高さ、幅で床掘する。
2. Install a drainage pump so that rainwater due to rainfall does not stay in the floor moat.
   降雨により床堀個所に雨水が滞水しないようにポンプなどで排水する。
3. For the foundation crushed stone, use the aggregate of the particle size specified in the specifications.
   基礎砕石は設計図書に示された粒度の骨材を使用する。

4. Compact the foundation crushed stone sufficiently with a compaction machine such as a tamper.
   基礎砕石はタンパ等の転圧機械で十分に転圧しなければならない。
5. When assembling the formwork, mark the reference height of each station inside the formwork so that the height of the top of the foundation concrete is within the control range.
   型枠の組立時には、各測点の基準高さを型枠内へマーキングして面木を取り付けたり目印釘により基礎コンクリートの天端高さが管理範囲内に収まるよう施工する。
6. Fix the wooden formwork to the formwork with square pipes or pipe supports so that the boards of wooden formwork do not stick out or collapse during casting concrete.
   木製型枠はコンクリート打設時にせき板がはらんだり崩れたりしないように、型枠には角パイプやパイプサポートで固定する。
7. Apply a release agent to the inside of the formwork before ready-mixed concrete is poured, to prevent concrete from adhering when the formwork is removed.
   コンクリート打ち込み前に型枠内に剥離剤を塗布し、型枠取り外し時のコンクリート付着を防止する。
8. When casting ready-mixed concrete, if there is water retention in the formwork, drain it with a pump.
   コンクリート打ち込み時、型枠内に滞水があればポンプ等で排水する。
9. The ready-mixed concrete should be cast in promptly, and the finish will be a wooden trowel finish.
   コンクリートの打ち込みは速やかに行い、仕上げは木コテ仕上げとする。

photo example : Construction status of foundation crushed stone paving

photo example : Installation of foundation concrete formwork

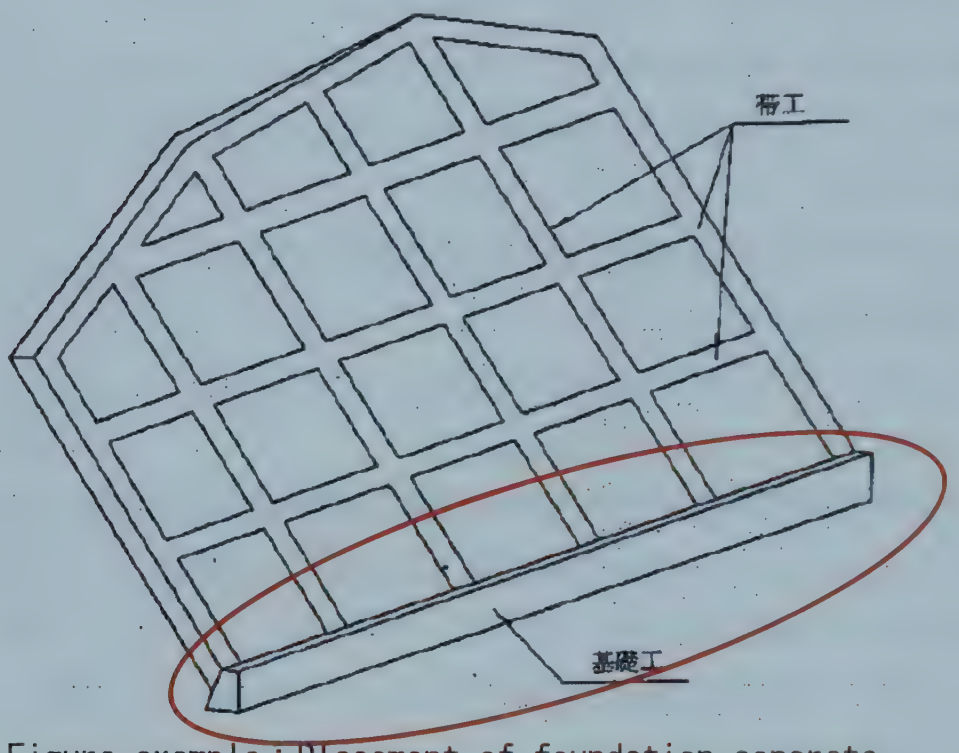


Figure example : Placement of foundation concrete

④ Lath netting work 金網設置

1. Lay the lath net from the top to the bottom of the slope so that it fits well on the uneven surface.
   金網を斜面の上から下へ凹凸になじみよく張り付ける。
2. Fix the lath net with anchor pins to prevent the lath net from sagging or moving.
   金網がダレたり移動しないようにアンカーピンで固定する。
   The number of anchor pins to be driven shall be the following number or more.
   - Main anchor pin (ϕ16mm L = 400mm) 30pcs / 100m2

・ Sub anchor pin（φ9mm L = 200mm）150pcs / 100m2

アンカーピンの打ち込み本数は下記の本数以上が望ましい。

・メインアンカーピン（φ16mm L=400mm）　30pcs/100m2

・補助アンカーピン（φ9mm L=200mm）　150pcs/100m2

3. When connecting the wire mesh, the minimum lap width should be 100 mm or more so that there is no gap in the lath net.

金網を繋げる際は、金網に隙間ができないように最低重ね合わせ幅を 100 mm以上とする。

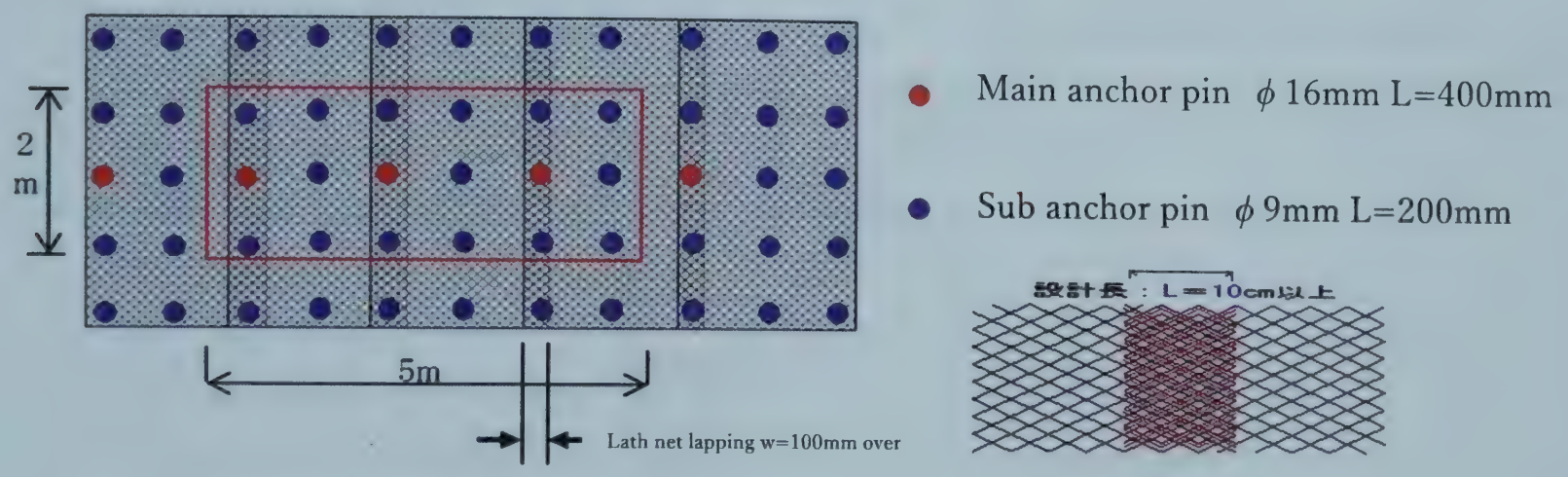


Figure example：Lath net installation standards

⑤ Scaffolding 足場組立

1. Assemble rebar assembly, formwork assembly and scaffolding for casting concrete. (catwalk)

鉄筋組立、型枠組立及びコンクリート打ち込み用の足場を組み立てる。（キャットウォーク）

2. Since loading of heavy materials or equipment on the catwalk is not permitted, it is necessary to install a separately structurally calculated steel tube slope scaffolding if a place for loading heavy objects is required on the slope.

キャットウォークへの重量物の積載は認められないため、法面上へ重量物の載荷場所が必要な場合は、別途構造計算された単管傾斜地足場の設置が必要となる。

3. Elevating equipment will be extended according to the construction of the scaffolding

足場の施工に合わせて昇降設備も延伸する。

photo example : Scaffolding on the slope

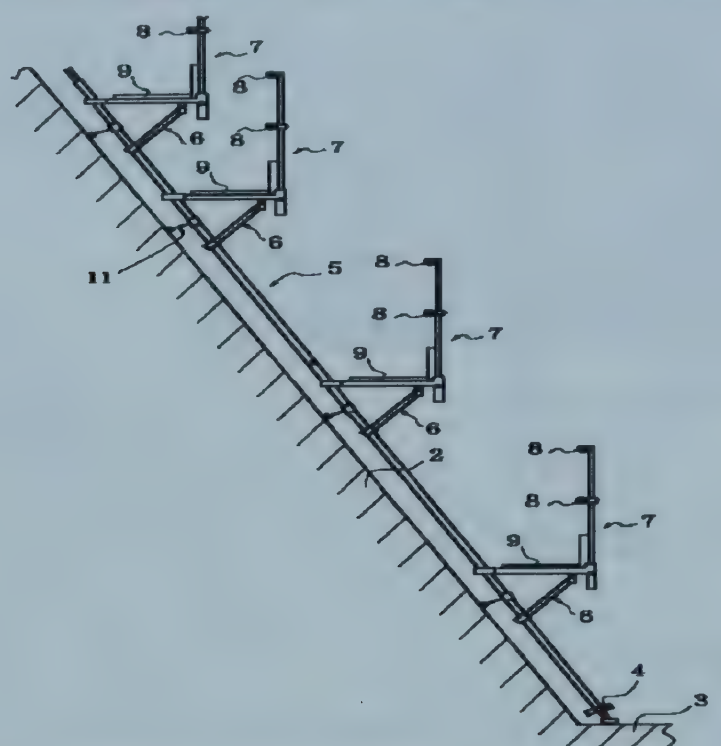


Figure example : Work scaffolding on the slope

⑥ Reinforcement / Formwork　鉄筋組立・型枠組立

【Reinforcement 鉄筋組立】

1. Clean the re-bar before assembling to remove any rust and other material that could interfere with the adhesion of the re-bar to the ready-mixed concrete.
   鉄筋は組み立てる前に清掃し、浮きさび、その他鉄筋とコンクリートとの付着を害する恐れのあるものを取り除く。
2. Mark the position of the reinforcing bar on the slope with a thin rope, etc., and assemble re-bar according to the dimensions shown in the drawings.
   法面に鉄筋の配置位置を PP ロープなどでマーキングし、設計図書に示された寸法通りに組み立てる。
3. Care should be taken when moving the material to the slope and handling the re-bar so that the re-bar is not deformed.
   材料の運搬、法面への楊重は鉄筋が変形しないように取り扱いを十分注意する。

4. Assemble the reinforcing bars so that the planned Spacing beams of the crib and covering depth be secured.
鉄筋の組み立ては計画された法枠中心間隔やコンクリートの被り厚さが確保できるように組み立てる。
5. Place spacers at appropriate positions and intervals to ensure proper re-bar covering depth.
鉄筋の被り厚さを正しく保つために、スペーサーを適切な位置と間隔で配置し、確実に固定する。
6. Reinforcing bar joints shall not be installed in the same position on the frame. Since an intersection of frames is a place that is susceptible to stress, it should not be installed at an intersection.
鉄筋の継手位置は同じ位置に設けず相互にずらす。法枠交点は応力を受けやすい個所となることから、その位置を外す。
7. The lap splicing length of the reinforcing bar shall be longer than the planed lap splicing length required by the standards (ASTM, JIS, BS, etc.).
鉄筋の継手長さは基準（ASTM、JIS、BS等）で求められた基本定着長以上とする。
8. Reinforcing bars are tied with a binding wire. No welding connection is made.
鉄筋は結束線で結束する。溶接接続はしない。
9. Connect the main anchor bar and sub anchor bar to the main reinforcement so that the assembled rebar does not move. The number of main anchor bars and sub anchor bars to be installed should follow the drawings.
組み立てた鉄筋が移動しないように主アンカーバー、補助アンカーバーを主鉄筋と連結する。主アンカーバー、補助アンカーバーの打設本数は設計図書に従う。

【Formwork 型枠組立】

10. The formwork material should be the one that has the required strength and composition while accurately ensuring the dimensions, spacing and shape of the concrete formwork.
型枠材料は、コンクリート法枠の寸法、間隔及び形状を正確に確保するとともに、所要の強度、合成を有するものを使用する。
11. The separator shall not be welded to the main reinforcement.
セパレーターは主鉄筋に溶接しない。
12. If there is a gap at the lower end of the formwork due to the unevenness of the slope surface, add a form panel or fill the space to prevent leakage from the gap between the formwork and the ground when placing ready-mixed

concrete.
法面地山の凹凸により型枠下端部に隙間ができる場合は、せき板を継ぎ足したり、間詰めをし、コンクリート打設時に型枠と地山の隙間から漏出しないよう処置する。

13. If the angle of the slope exceeds 20 degrees, cover the front of the legal frame with a form panel.
A part of the opening will be provided so that ready-mixed concrete can be driven and compacted by a vibrator, and a lid will be provided at the time of casting.
斜面の勾配が1：3.0を超える場合は、法枠正面についてもせき板で蓋をする。
コンクリートの打ち込みとバイブレーターによる転圧ができるよう、一部開口部を設け、打設時に蓋をする。

photo example：Formwork opening

14. The Timbering and support works use materials with sufficient bearing capacity and stability.
支保工は、十分な支持力と安定性を有する資材を使用する。
15. If using a pipe support, install it parallel or perpendicular to the frames of crib.
パイプサポートを使用する場合は、法枠に対し平行または垂直に取り付ける。
16. After assembling the formwork, measure the dimensions inside the formwork, the thickness covering the reinforcing bars, and the interval between the formwork, and if there are any abnormalities, redo the formwork.
型枠組立後は、枠内の寸法、鉄筋の被り厚さ、法枠の間隔を測定し、異常がある場合は型枠をやり直す。

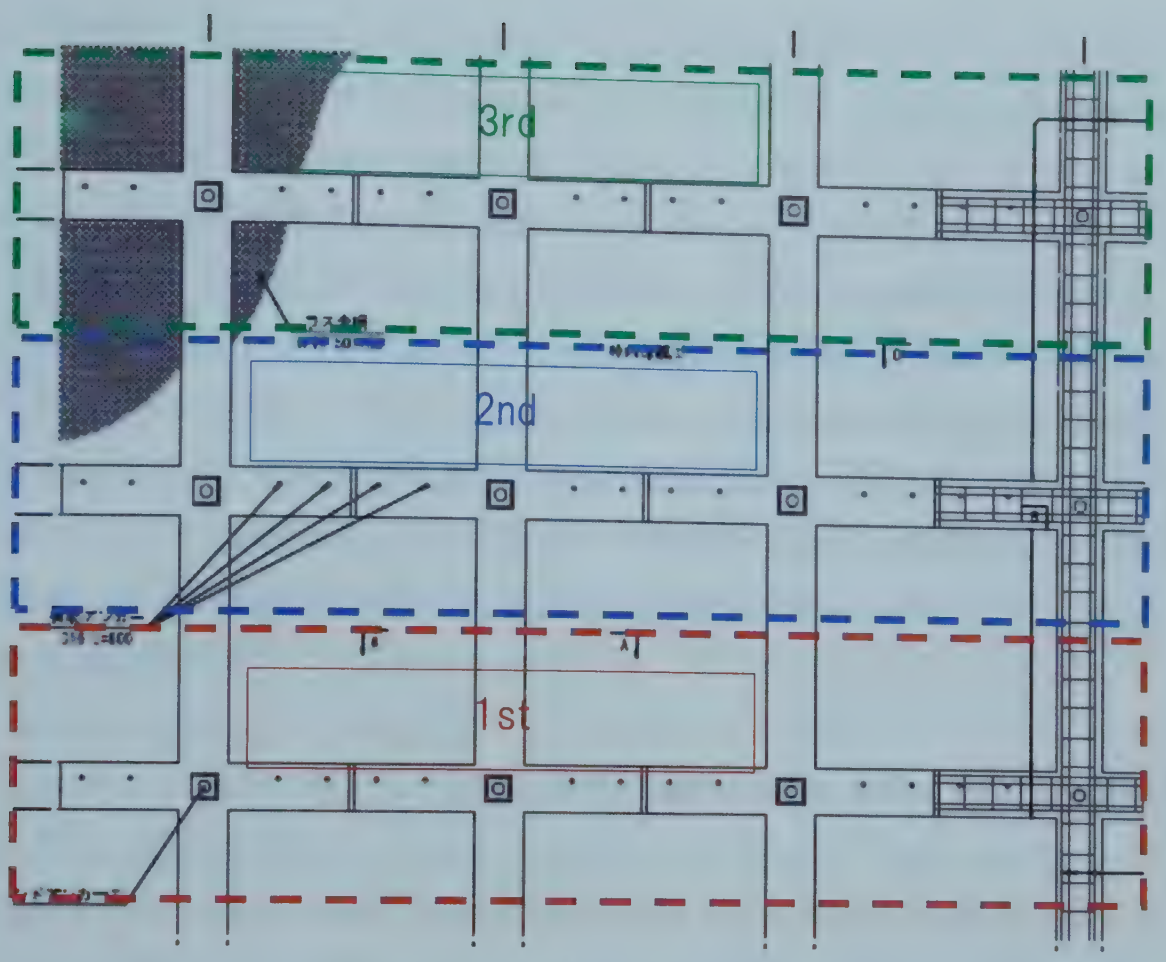


Figure example : work order

⑦ Cast concrete コンクリート打設

1. Before casting ready-mixed concrete, apply the form oil evenly to the inner form and prevent the form oil from adhering to the reinforcing bars.
   コンクリートの打ち込み前に型枠内面へ剥離剤を均一に塗布するとともに、剥離剤が鉄筋に付着しないようにする。
2. The concrete used for the concrete crib shall be formulated with the unit water content as small as possible after satisfying the compounding conditions.
   法枠に使用するコンクリートは、配合条件を満足したうえで、単位水量をできるだけ少なくした配合とする。
3. When transporting ready-mixed concrete at a concrete manufacturing plant, transport it with an agitator body truck.
   When a dump truck or flat body truck is used, the segregation and the quality will deteriorate.
   コンクリート製造工場で練り混ぜたコンクリートを運搬する場合は、トラックアジテータで運搬する。ダンプトラックや平ボディートラックなどを使用した場合、コンクリートが材料分離し品質が低下する。
4. The ready-mixed concrete should be transported promptly, casted and compacted immediately.

コンクリートは速やかに運搬し、ただちに打ち込み、十分に締固めする。

5. The time from the production of ready-mixed concrete to the casting should be within 1.5 hours when the outside temperature exceeds 25℃, and does not exceed 2 hours when the outside temperature is 25℃ or less.
練り混ぜてから打ち込むまでの時間は、外気温が25℃を超える場合は1.5時間以内、25℃以下の場合は2時間を超えない。
6. Prior to pumping ready-mixed concrete, mortar similar to mortar in ready-mixed concrete should be pumped so that the mortar in ready-mixed concrete does not adhere to the pump or the like nor decrease while transportation. Also, this mortar should not be cast into the formwork. (blinding concrete)
コンクリートの圧送に先立ち、コンクリート中のモルタルと同程度のモルタルを圧送し、コンクリート中のモルタルがポンプ等に付着して少なくならないようにする。また、そのモルタルは型枠内には打ち込まない。(捨てモル)
7. As a rule, the ready-mixed concrete input height should be 1.5m or less, and the casting position should be moved appropriately so that material segregation does not occur.
コンクリートの投入高さは、原則として1.5m以下とするとともに、打ち込み位置を適宜移動させ、材料分離が生じないよう施工する。
8. Ready-mixed concrete is compacted using a vibrator.
コンクリートはバイブレーター等を用いて密実なコンクリートとなるよう十分締め固める。
9. The standard height of one layer of concrete is 40cm to 50cm or less, and the insertion interval of the vibrator is 50cm or less as standard.
コンクリートの一層の打ち込み高さは40 cm～50 cm以下とし、振動機の挿入間隔は50 cm以下を標準とする。
10. Do not touch the rebar that assembled the vibrator.
バイブレーター等を組み立てた鉄筋に直接当てて締め固めてはならない。
When ready-mixed concrete is cast into two or more layers, the upper layer ready-mixed concrete should be cast before the lower layer ready-mixed concrete begins to solidify, and a vibrator should be inserted into the lower layer ready-mixed concrete by about 10cm, to integrate the upper layer and lower layer concrete.
コンクリートを二層以上に分けて打ち込む場合、上層のコンクリートの打ち込みは、下層のコンクリートが固まり始める前に行い、振動機を下層のコンクリート中に10 cm程度挿入し、上層と下層のコンクリートが一体となるように施工する。

11. At the joint of concrete, poor quality concrete such as laitance should be removed, the surface should be roughened by chipping, and the surface should be sufficiently absorbed with water.
コンクリートの打ち継ぎ目は、打ち込み面のレイタンス等品質の悪いコンクリートを取り除き、チッピング等により面を粗にしたうえで、十分吸水させる。
12. After casting ready-mixed concrete, cure it properly so that it will not be affected by harmful effects such as low temperature, high temperature, sudden temperature change, drying, loaded, impact, and vibration.
コンクリートの打ち込み後、低温、高温、急激な温度変化、乾燥、荷重、衝撃、振動等の有害な作用の影響を受けないよう十分に養生する。

⑧ Form removal 型枠脱型

1. The formwork should be removed after confirming that the concrete has hardened sufficiently.
Generally, the form panel may be removed after 6 days or more when the average temperature is 10℃ or more and less than 20℃, and 4 days or more when the average temperature is 20℃ or more.
型枠の脱型時期はコンクリートが十分に固まったことを確認した後行う。
一般的に平均気温が 10℃以上 20℃未満の場合は 6 日以上、20℃以上の場合は 4 日以上経過すれば型枠を取り外してもよい。
2. Carefully remove the formwork. The order of removal is to remove the vertical beams(frames), which are relatively difficult to receive the load, and then remove the horizontal beams(frames).
型枠の取り外しは慎重におこなう。取り外す順序は比較的荷重を受けにくい縦梁を外し、次に横梁を取り外す。
3. Post-filling of the separator cone is treated with non-shrink mortar.
セパレーターコーンの後埋めは、原則として無収縮モルタルで処理する。

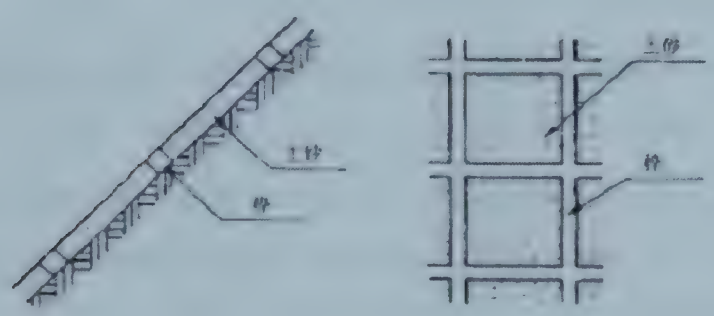
⑨ Remove scaffolding 足場解体・盛替え

1. Dismantle the work scaffolding where the cast concrete and formwork removal have been completed and replace it on the upper slope.
コンクリートの打設と型枠脱型が完了した個所の作業用足場は解体して上部へ盛り替える。

※ Repeat steps ⑤ to ⑨ until the concrete crib work is completed.
⑤～⑨の作業を法枠が完成するまで繰り返す

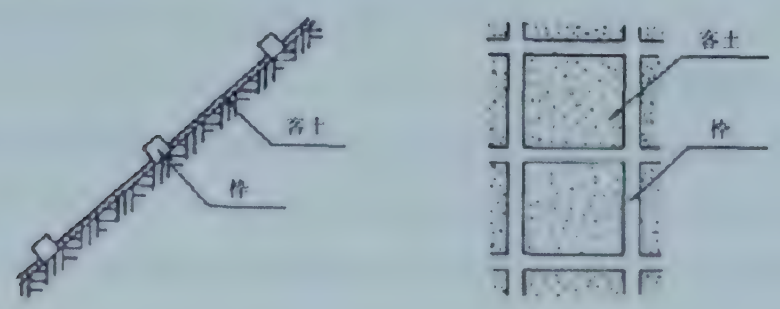
⑩　中詰材設置

1. コンクリート枠内の中詰工の設計は、現場条件、工事目的等を考慮し設計する。
   下記に選定項目の例をあげる。
   1）盛土法面か切土法面か
   2）緑化が必要であるか
   3）法面勾配、岩質、地質条件
   4）気象条件
   5）湧水、集水状況
2. 中詰工は主に風化防止、浸食防止を目的としているが、工法は次のような種類がある。
   1）土砂詰工（緑化）　枠内に土砂を詰める工法



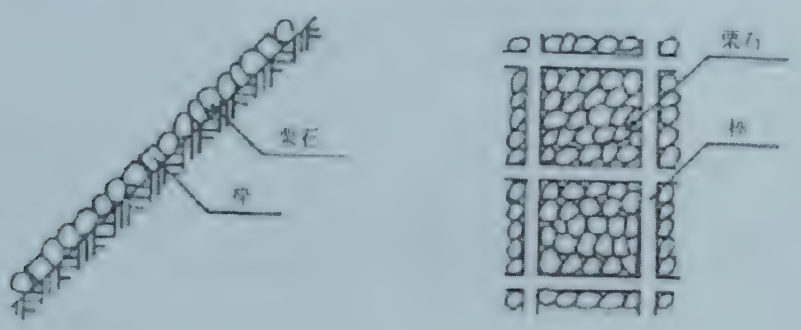
   2）土のう積工（緑化）　土のうに土砂を詰め枠内に敷きならべる工法

   3）客土吹付工（緑化）　枠内に種子・堆肥などを混入した客土を 1cm～3cm 程度の厚さで吹き付ける工法

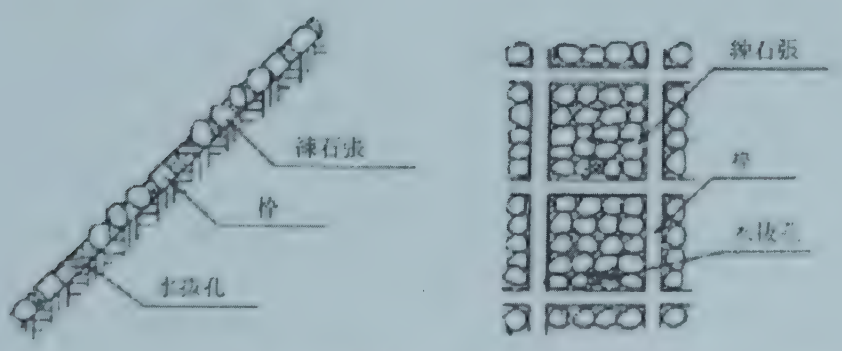


   4）植生基材吹付工（緑化）　枠内に緑化用機材を 5 cm～10 cm程度の厚さで吹き付ける工法

5） 空石張工（湧水・集水、防草）　枠内に栗石、粗石を詰める工法



6） 練石張工（防草）　枠内に栗石、粗石などを詰め、間隙をコンクリートで充填する工法



7） 平板ブロック張工（防草）　枠内にプレキャストコンクリート板を敷きならべる工法

8) コンクリート張工(防草)　枠内に現場打ちコンクリート(またはモルタル吹付)を打設する工法

⑪ 完成

1. 完成した法枠の縦梁、横梁すべてを測量し完成図を作成する。

Equipment and machinery 使用機械

一般的な使用機械は以下の通りである。

Equipment	Specification	
Excavator	0.2m3～0.7m3	
Bulldozer		
Transporting vehicle on rough terrain		
Dump truck	3t～10t	
Bar cutter		
Bar bender		
Buzz saw		
Concrete bucket	0.5m3～	
Concrete vibrator		
Concrete pomp		
Agitator body truck		
Crane		

Temporary and Preparatory works 仮設計画

1. Work rope (master main rope) 作業用ロープ（親綱）

   Treatment of uneven surface, Removal of rock and boulders, and Crib frame positioning work are performed by attaching a special safety slope belt to the work rope.

   基面整形、浮石除去、法枠の位置出し作業は作業用ロープに専用の安全帯を取り付けて作業する。

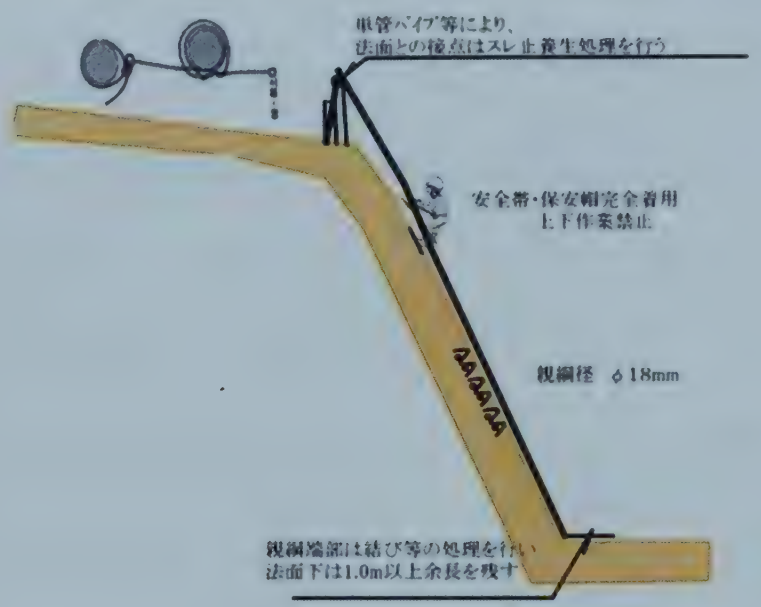


Figure example : Installation of work rope

2. Scaffolding 作業用足場

   Work scaffolding will be installed for assembling reinforcing bars and formwork and casting concrete.

   鉄筋・型枠の組み立て、コンクリートの打設のため作業用足場を架設する。

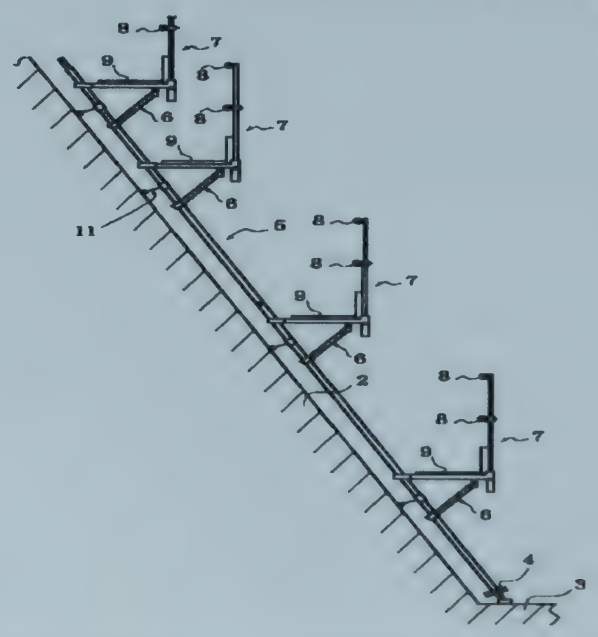


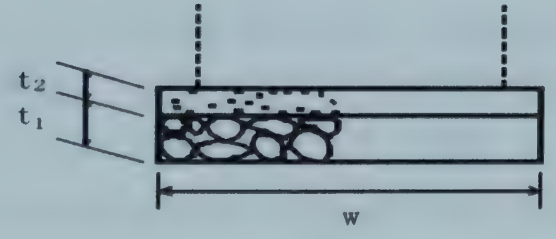
Figure example : Scaffolding for slope

Work progress management and Quality control 施工管理

Work progress management 出来形管理

W1: Foundation stone inspection 基礎砕石出来形

1. Measure the width, thickness and length of the foundation stone.
   基礎砕石の幅、厚さ、延長を測定する。
2. The measurement frequency is 1 position for every 40m of construction length or 2 positions for station distance of 40m or less.
   測定頻度は施工延長 40m につき 1 箇所または施工延長 40m 以下のものは 2 箇所。
3. The tolerances for work are as follows.
   Width: More than the design value.
   Thickness: within -30mm
   Extension: within -200 mm
   規格値は次の通りである。
   幅：設計値以上
   厚さ：-30mm 以内
   延長：-200mm 以内



Figure：Measurement point

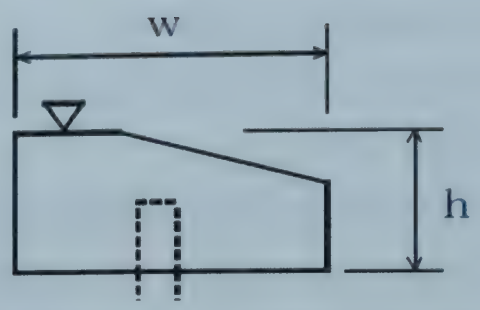
W2:Foundation concrete inspection 基礎コンクリート出来形

1. Measure the width, thickness and length of the foundation concrete.
   基礎コンクリーの基準高さ、幅、高さ、延長を測定する。
2. The measurement frequency is 1 position for every 40m of construction length or 2 positions for station distance of 40m or less.
   測定頻度は施工延長 40m につき 1 箇所または施工延長 40m 以下のものは 2 箇所。
3. The tolerances for work are as follows.
   Elevation: ± 30mm
   Width: within -30mm
   Height: within -30mm
   Distance: within -200mm
   規格値は次の通りである。
   基準高さ：±30mm

幅：-30mm 以内

高さ：-30mm 以内

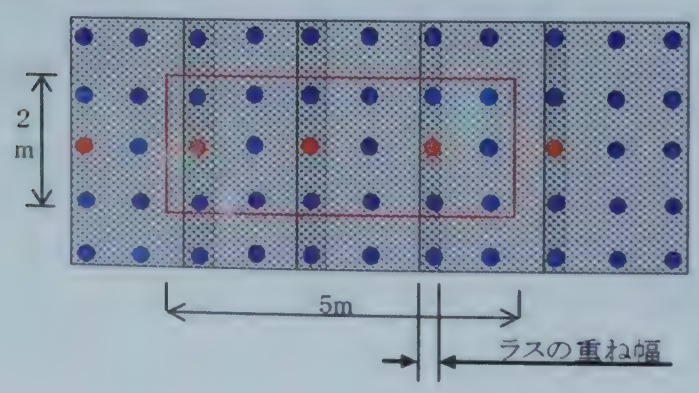
延長：-200mm 以内

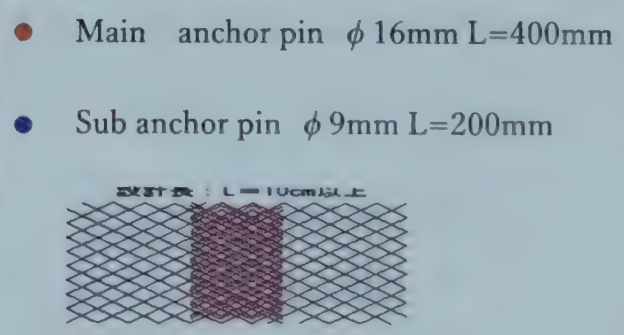


Figure：Measurement point

W3:Lath net Inspection 金網設置の確認

1. After the lath net is well laid on the slope and if irregularities are found, check that it is fixed with anchor pins so that it is as close to the ground as possible.
   法面へなじみよく張り、凹凸がある場合は出来る限り地山に密着するようにアンカーピンで固定してあるか確認する。
2. Check if the number of anchor pins to be fixed are 3 main anchor pins / 10m2 and 15 sub anchor pins / 10m2 or more.
   固定するアンカーピンの本数は、主アンカーピン　3 本/10m2、補助アンカーピン 15 本/10m2 以上になるように配置して打ち付けてあるかどうかを確認する。
3. Check if the lath net seams are woven or tied with binding wire.
   金網の継ぎ目が編み込まれてあるかまたは結束線で結束されているかどうかを確認する。
4. The overlapping width of the lath net is 10cm or more.
   金網の重ね合わせ幅は 10cm 以上。
5. Frequency of inspection is 1 position at every 200m2.
   確認頻度は 200m2 毎に 1 箇所。





W4:Scaffolding inspection 足場出来形

1. The measurement points of the scaffold are distance, upper width, and heigh. All steps should be measured.
   足場の測定個所は、延長、上幅、高さとし、全ての段数を測定する。

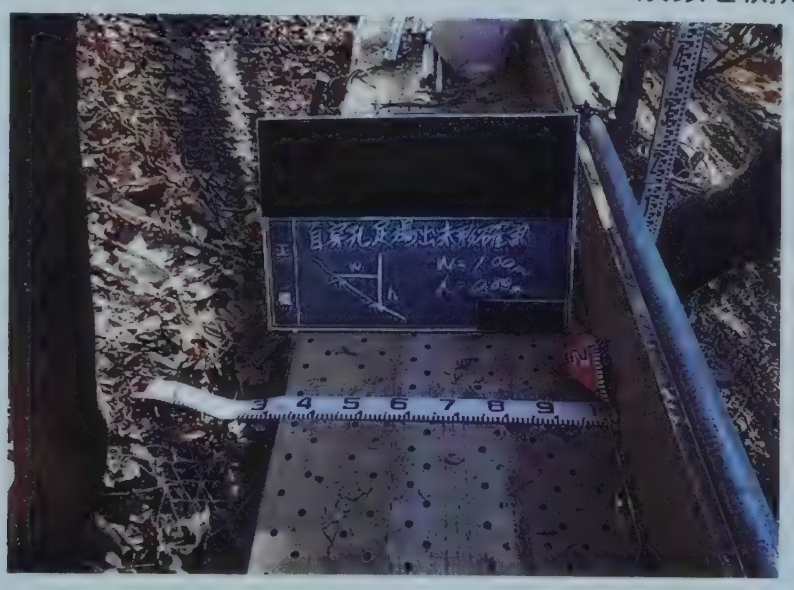


Photo example : Scaffolding inspection

W5: Spacing, Diameter, Lap splicing, Number of bar 鉄筋の配置・鉄筋径・鉄筋の本数・鉄筋の継ぎ手長さ

1. After assembling the reinforcing bars, inspect whether the reinforcing bars are arranged according to the specifications according to the design or not.
   鉄筋の組み立て完了後に設計図書に沿った仕様で鉄筋が配置されているかどうかを確認する。
2. Inspect the diameter of the placed reinforcing bars and the number of main reinforcing bars required in the cross section.
   配置された鉄筋の径、断面で必要な主鉄筋の本数を確認する。
3. Make sure that the joints of the main reinforcing bars are not concentrated on one section and are not located at the intersections of the crib frames.
   主鉄筋の継ぎ手部分については一断面に集中しないように相互にずれていることかつ、法枠交点に配置されていないことを確認する。
4. Inspect whether the joint lap splicing of the main reinforcing bar is longer than the planed lap splicing length calculated according to the standard or not.
   主鉄筋の継ぎ手長さは基準に従って算出された基本定着長以上となっているかどうかを確認する。
5. Frequency of inspection is 1 position at every 200m of crib frames length.
   確認頻度は梁長 200m 毎に 1 箇所。

W6: Spacing frames of the crib(before cast ready-mixed concrete)法枠の間隔（コンクリート打ち込み前）

1. Measure and inspect that the spacing between the reinforcing bar intersections is as planned.
   鉄筋交点の間隔が、計画された寸法通りか測定して確認する。
2. Frequency of inspection is 1 position at every 200m of crib frames length.
   確認頻度は梁長 200m 毎に 1 箇所

W7: Inside dimensions of assembled forms (Width, Height)型枠内寸法断面の幅と高さ（コンクリート打ち込み前）

1. Measure and inspect that the section of the inner dimension of the formwork is as planned.
   型枠の内側寸法の断面が、計画された寸法通りか測定して確認する。
2. Frequency of inspection is 1 position at every 200m of crib frames length.
   確認頻度は梁長 200m 毎に 1 箇所。

W8: Covering depth for concrete and main reinforcement 鉄筋のコンクリートの被り厚さ（型枠と鉄筋の間隔）

1. Measure the distance between the main reinforcing bar and the inside edge of the formwork, which is the end of the concrete.
   主鉄筋とコンクリートの端部となる型枠内側の間隔を測定する。
2. Frequency of inspection is 1 position at every 200m of crib frames length.
   確認頻度は梁長 200m 毎に 1 箇所。

W9: Number and spacing for Main anchor bar and Sub anchor bar 主アンカーバー・補助アンカーバーの本数

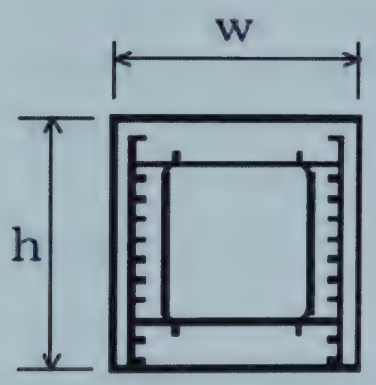
1. Make sure that the main anchor bars are installed so as to fix the main reinforcing bars at the frame intersections. Also check if they are placed at each intersection.
   主アンカーバーが枠交点の主鉄筋を固定するように撃ち込まれてあるかを確認する。各交点に配置されているかについても併せて確認する。
2. Check if the number of sub anchor bars according to the design is arranged on the horizontal frames. Also, check whether the sub anchor bar is installed so that the horizontal frames does not sag.
   補助アンカーバーが設計図書に準じた本数が横梁に配置されているかどうかを確

認する。また、横梁にたるみがないように打設されているかどうかも確認する。

3. Frequency of inspection is 1 position at every 200m of crib frames length.
   確認頻度は梁長 200m 毎に 1 箇所。

W11: Width and Height for frames of Crib 法枠断面の幅・高さ（完成出来形）

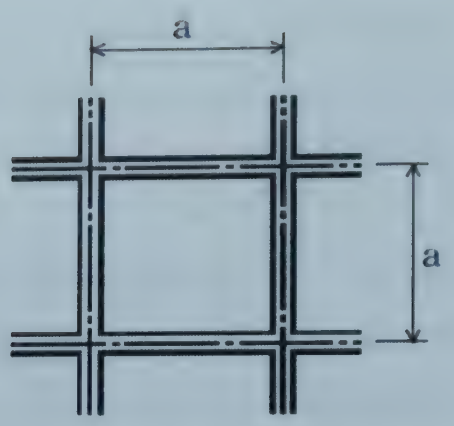
1. Measure the width and height of the completed concrete frames.
   完成したコンクリート法枠の幅と高さを測定する。
2. Frequency of inspection is 1 position at every 100m of crib frames length.
   確認頻度は枠延長 100m につき 1 箇所。
3. The tolerances for work are as follows.
   Width: within -30mm
   Height: within -30 mm
   規格値は次の通りである。
   幅：-30mm 以内
   高さ：-30mm 以内



Figure：Measurement point

W12: Spacing frames of crib 法枠中心間隔（完成出来形）

1. After the concrete crib is completed, measure and confirm whether the center spacing of the crib frame conforms to the design.
   法枠の完成後に法枠の中心間隔が設計図書に準じているか測定して確認する。
2. Frequency of inspection is 1 position at every 100m of crib frames length.
   確認頻度は枠延長 100m につき 1 箇所、枠延長 100m 以下は 2 箇所。
3. The tolerances for work are as follows.
   Frame center spacing: ±100mm
   規格値は次の通りである。
   枠中心間隔：±100mm



Figure：Measurement point

W13：Inspection of slope length 法長の測定（完成出来形）

1. Measure and inspect that the slope length of the station conforms to the design.
   測点断面の法長が設計図書に準じているかを測定して確認する。
2. Frequency of inspection is 1 position at every 40m of station distance.
   確認頻度は施工延長 40m につき 1 箇所。
3. The tolerances for work are as follows.
   L＜10m：-100 mm
   L≧10m：-200 mm
   規格値は以下の通りである。
   L＜10m：-100 mm
   L≧10m：-200 mm

Quality control 品質管理

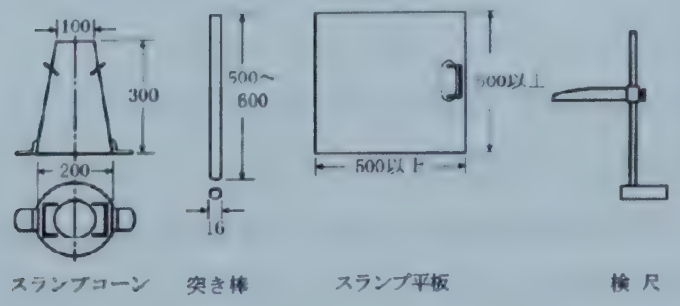
Q1: Concrete design mixed proportion コンクリートの配合

1. The concrete mixed ratio must satisfy the design strength according to the specifications.
   コンクリートの配合は仕様に準じた設計強度を満足しなければならない。
2. The compounding proportion of concrete is approved by submitting a compounding plan to the Engineer in advance.
   コンクリートの配合は事前に監督職員へ配合計画書を提出し承認を受ける。
3. The following tests are conducted in advance for the concrete composition and the consent of the Engineer should be obtained. When using aggregates, cement, etc. of the same quality as those recently tested, the test can be omitted with consent.
   コンクリートの配合は事前に以下の試験を行い監督職員の承諾を受ける。ただし、最近試験を行ったものと同じ品質の骨材、セメント等を使用する場合は承諾を受けて試験を省略することができる。
   1) Decantation test (sieve analysis) for aggregate 骨材ふるい分け試験 JIS A1102 or Indian or international equivalents
   2) Slump test スランプ試験 JIS A 1101 or Indian or international equivalents
   3) Air content test 空気量試験 JIS A 116, 118, 1128 or Indian or international equivalents
   4) Test for compressive strength 圧縮強度試験 JIS A 1108 or Indian or international equivalents
   5) Test for chloride content 塩化物含有量試験 JIS A 1144 or Indian or international equivalents
   6) Test for alkali-aggregate reaction 骨材のアルカリシリカ反応性試験 JIS A 1145, 1146 or Indian or international equivalents
   7) Test for water mixed ratio 単位水量試験

Q2:Slump test of ready-mixed concrete コンクリートのスランプ試験

1. When receiving ready-mixed concrete, a slump test will be conducted to confirm the fluidity of the ready-mixed concrete.
   コンクリート荷受け時に、コンクリートの流動性を確かめるためスランプ試験を実施する。
2. Frequency of inspection is once / day or more, or once every 150m3.
   確認頻度は、1回/日以上または、150m3毎に1回。

3. The tolerances for work are as follows.
   Set slump value 5cm or more and less than 8cm: ±1.5cm
   Set slump value 8cm or more and 18cm or less: ±2.5cm
   規格値は次の通りである。
   設定スランプ値 5 cm以上 8 cm未満：±1.5 cm
   設定スランプ値 8 cm以上 18 cm以下：±2.5 cm



Figure：Slump tester (JIS A 1101)

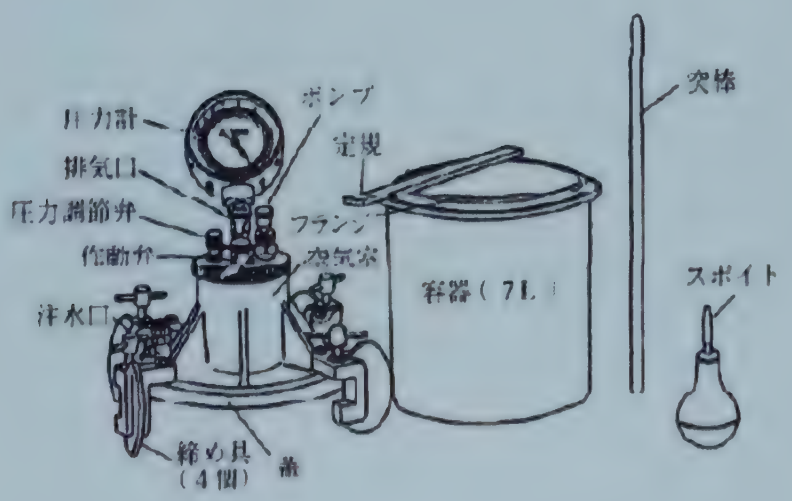
Q3: Test for compressive strength of concrete コンクリートの圧縮強度試験

1. Create a test piece when the ready-mixed concrete is accepted at the site.
   Create the number of test pieces according to the specifications.
   コンクリートの現場受け入れ時に合わせて、供試体を採取する。
   供試体の本数は仕様書に準じた本数を採取する。
2. The size of the test piece is manufactured with a sampler of φ100mm L=200mm if the design size of the coarse aggregate is up to 25mm, and with a sampler of φ125mm L=250mm if the size of the coarse aggregate is 40mm.
   供試体のサイズは、粗骨材の設計寸法が 25mm までであればφ100mm L=200mm の圧縮用型枠器で採取し、粗骨材の寸法が 40mm であればφ125mm L=250mm の圧縮用型枠器で採取する。
3. The test method conforms to JIS A 1132 or Indian or international equivalents.
   試験方法は JIS A 1132 に準ずる。
4. Frequency of inspection is once / day or more, or once every 150m3.
   確認頻度は、1 回/日以上または、150m3 毎に 1 回。
5. The tolerances value is that the result of one test is 85% or more of the specified nominal strength, and the average value of the results of three tests is equal to or more than the specified nominal strength.
   規格値は、1 回の試験結果は指定した呼び強度の 85%以上で、3 回の試験結果の平均値は指定した呼び強度以上であること。

photo：concrete sampler

Q4: Air content test of ready-mixed concrete コンクリートの空気量測定

1. Measure the amount of air in the ready-mixed concrete when receiving the concrete.
   コンクリートの荷受け時にコンクリート中の空気量を測定する。
2. The test method conforms to JIS A 1116, 1118, 1128 or Indian or international equivalents.
   試験方法はJIS A 1116, 1118, 1128に準ずる。
3. Frequency of inspection is once / day or more, or once every 150m3.
   確認頻度は、1回/日以上または、150m3毎に1回。
4. The tolerances value differs depending on the type of concrete, but it is 4.5% ± 1.5% for ordinary concrete.
   規格値は、コンクリートの種類により違いがあるが、普通コンクリートであれば4.5%±1.5%となっている。



Figure：tester

Q5:Materials 使用材料

1. Prior approval should be obtained from catalogs, copies of mill sheets, etc. before bring-in of materials to be used on site. Only approved materials can be used in the site.
   現場で使用する予定の材料を搬入前にカタログ、ミルシートの写し等で事前承認を

受ける。現場では承認された材料のみ使用することができる。

2. Inspect the specifications and quantity of the materials brought into the site, and confirm that they match the arrival slip.
   現場に搬入された材料の規格、数量を確認し、入荷伝票と合致しているか確認する。
3. Immediately submit the mil sheets or test reports of the received materials to the Engineer.
   入荷した材料の試験成績表も直ちにエンジニアへ提出する。
4. Visually check the aggregate for mud, shells, plant roots, etc.
   If foreign matter is mixed in, the strength of concrete may decrease.
   骨材については、泥、貝殻、植物の根などが混入していないか目視にて確認する。
   混入されていると強度低下になるおそれがある。

METHOD STEATEMENT

施工計画

施工管理

仮設計画

HORIZONTAL DRAINAGE BORING WORKS

Work Flow 施工フロー	Work progress 出来形管理	QA/QC 品質管理
① Preparation of slopes 準備工		
② Excavation, Trimming and Clearing 基面処理工		
③ Foundation concrete 基礎コンクリート	W1:foundation stone 基礎砕石（均しコンクリート）出来形	Q1:Concrete design mixed proportion コンクリート配合計算書
	W2:foundation concrete 基礎コンクリート出来形	Q2:Slump test コンクリートのスランプ試験
		Q3:Uniaxial compressive strength test コンクリートの圧縮強度試験
		Q4:Air volume test for ready-mixed concrete コンクリートの空気量測定
④ Lath netting work 金網設置	W3:lath net 金網設置・重ね幅の確認	Q5:Material 使用材料
	Working drawing	
⑤ Scaffolding 足場組立	W4:Scaffolding 足場出来形	
⑥ Reinforcement / Formwork 鉄筋組立・型枠組立	W5: Spacing, Diameter, Lap splicing, Number of bar 鉄筋の配置・鉄筋の径・鉄筋の本数・鉄筋の継ぎ手長さ	Q5:Material 使用材料
	W6:Spacing frames of the crib 法枠の間隔（交点）	
	Inside dimensions of assembled forms (Width, Height) 型枠内寸法断面の幅と高さ	
	W8:Covering depth for concrete and main reinforcement 鉄筋とコンクリートの被り厚さ（型枠と鉄筋の間隔）	
	W9:Number and spacing for Main anchor bar and Sub anchor bar 主アンカーバー・補助アンカーバーの配置・本数	
⑦ Cast concrete コンクリート打設		Q1: Concrete design mixed proportion コンクリート配合計算書
		Q2:Siump test コンクリートのスランプ試験
		Q3:Uniaxial compressive strength test コンクリ

		ートの圧縮強度試験
		Q4:Air volume test for ready-mixed concrete コンクリートの空気量測定
⑧ Form removal 型枠取り外し		
⑨ Remove scaffolding 足場解体・盛替え		
⑩ Filling material within frames 中詰材設置	中詰材出来形	
⑪ Complete 完成	W11:Width and Height for frames of Crib 法枠断面の幅・高さ	
	W12: Spacing frames of crib 法枠中心間隔	
	W13:Length of slopes 法長の測定	

WORK PLAN 施工計画

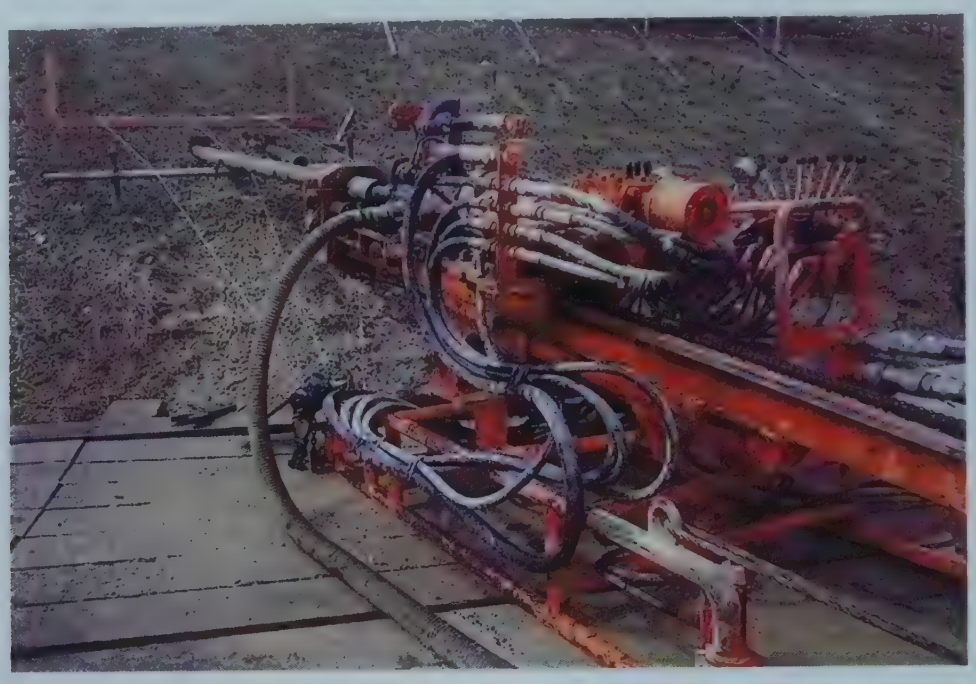
① [illegible] 準備工

1. Prior to Horizontal Drainage Borings work, we will investigate the site condition and check the design. If there is a difference in site condition between design and the actual site, report to the Engineer and ask instruction.
   Also, it is necessary to check available boring logs and core samples to know the position of the targeted aquifers in advance.
   施工に先立ち現地の現地状況の調査、設計の照査を行う。現地と設計内容に相違がある場合は、監督職員と協議する。
   また、柱状図、コアサンプルを照査し、帯水層の位置を事前に把握しておく必要がある。
2. Mark the drilling point and drilling direction at the site.
   There are two ways to drill boreholes: one is to remove groundwater radially from one place in the landslide area, and the other is to directly aim the aquifer. Since the drilling direction is important in both methods, survey line points are provided before and after the drilling point so that the drilling direction can be confirmed before drilling.
   削孔ポイントと削孔方向をマーキングする。
   削孔方向は、地すべり範囲に１箇所から放射状に地下水を排除する配置と、帯水層をダイレクトに狙う方向に配置する方法がある。
   どちらの方法も、削孔方向が重要になってくることから、削孔ポイントの前後に側線ポイントを設けて、削孔前に削孔方向の確認ができるようにしておく。

② Drilling 削孔

1. For drilling, rotary boring machine, rotary and percussion drill, horizontal boring machine, and other relevant types boring machines are used.
   Since the limit depth of drilling depends on the conditions such as drilling machine specifications, drilling diameter, planned drilling depth, and stratum, it is necessary to select an adequate drilling machine so as not to cause borehole bentness due to the marginal machine capacity.
   When using a stationary (skid type) drilling machine, scaffolding for drilling is required.
   削孔機械はロータリー式ボーリングマシン、ロータリーパーカッション式ドリル、

水平ボーリングマシンなどを使用する。
削孔機仕様、削孔径、削孔深度、地層などの条件で限界深度が異なるため、限界機械能力により、孔曲がりなどを発生させないよう、削孔機械を選定する必要がある。
定置式（スキッドタイプ）の削孔機を使用する場合は、削孔用足場が必要になる。



Photo：Rotary and Percussion Type Drilling Machine

Photo：Horizontal Boring Machine

2. When sampling of core sample for geological logging during drilling is required, selection of machine is generally limited to rotary boring machines or horizontal boring machines.
   The tip bit to be used is also limited to diamond bits or metal crown bits.
   削孔中に地層確認のためのサンプリングを行う場合、一般的にはロータリー式ボーリングマシンや水平ボーリングマシンに限定される。
   さらに先端ビットもダイヤモンドビットかメタルクラウンに限られてくる。
3. Install a drilling machine at the drilling point, and start drilling after confirming the drilling angle and drilling direction.

削孔機を削孔ポイントに設置し、削孔角度、削孔方向の確認を終えた後に削孔を開始する。

4. During drilling, check the volume of water discharged from the drilling hole and collect and observe the drilling slime to see if there is any difference in geological conditions of design, including boring logs at the site.
削孔中、孔口から吐出される水量をチェックすることや、削孔スライムを確認して設計図書に記載されている地盤条件に相違が無いか確認する。

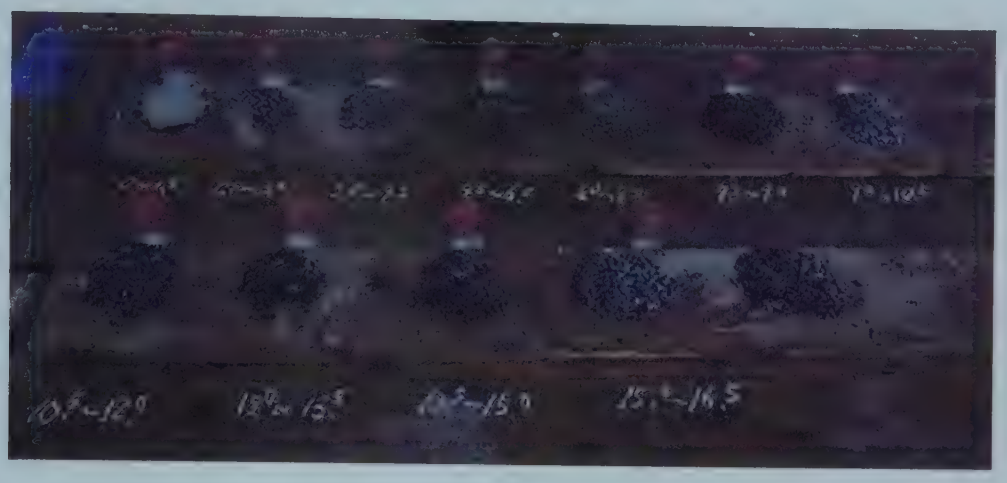


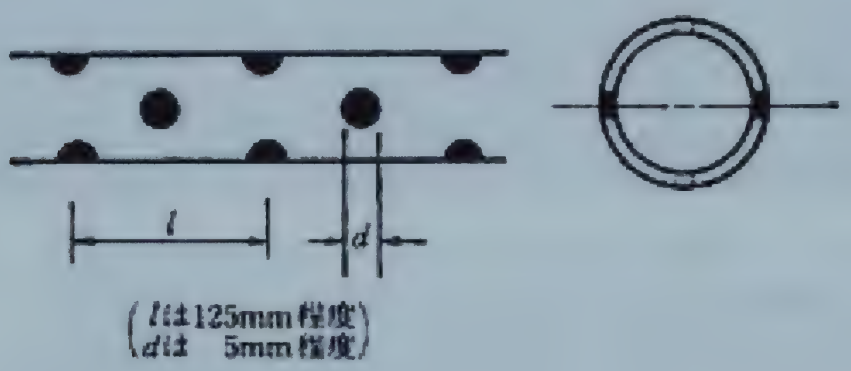
Photo: Slime Sampling During Drilling

5. Measure the drilling length immediately after drilling is completed.
削孔完了後は削孔長さを測定する。

③ Installation of the perforated pipes 保孔管挿入

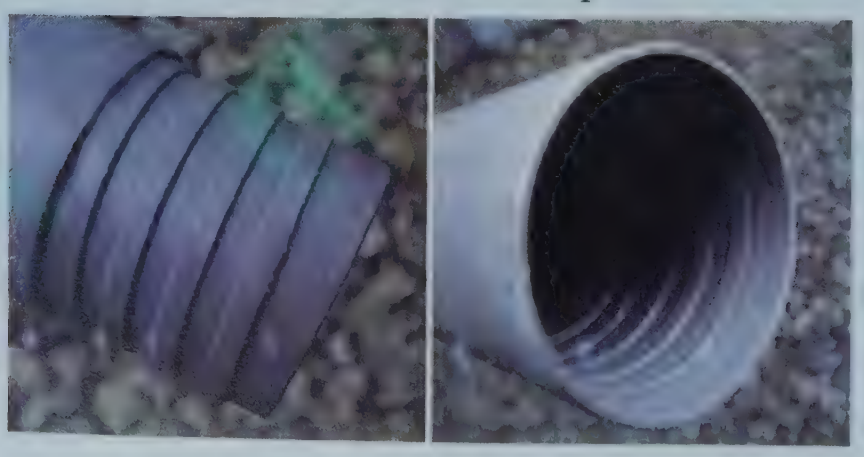
1. Use strainer-processed PVC pipes or steel pipes for the perforated pipes.
保孔管はストレーナー加工した硬質塩化ビニール管製か、鋼製を使用する。
2. If an insert type socket (DV joint, etc.) is used when connecting the perforated pipes, the socket part may get caught in the casing joint when inserting the perforated pipes, making it impossible to insert pipes. Also, the joint portion may come off and the perforated pipes may be left in the drilling hole.
Generally, internal screw type sockets are used, which keep outer diameter of perforated pipes constant even at socket sections. The direction of tightening screw should be the same as the direction of rotation of a drilling machine to be used.
保孔管を接続する際に、はめ込み式ソケット（DV 継手等）を使用すると、保孔管挿入時にソケット部分がケーシングジョイントに引っ掛かり、挿入できなくなることや、接続部が外れてしまい孔内に保孔管を残置してしまうことがある。
一般的には、内側ネジ加工式などを使用することが多い。ネジの締め回転方向は、選定した削孔機の回転方向と同じ方向で加工しなければならない。

3. Mark control numbers on the surface of each perforated pipe to install and have confirmation by supervisor / the Engineer of the total quantity of pipes before insertion.
保孔管毎に番号を記入し、挿入前に全数量の確認を受けておく。
4. After drilling is completed, pull out inner rods and insert the prepared perforated pipes one by one, while connecting one another.
Insert the pipes in the order of control numbers marked in the previous step and check that all the quantities have been inserted.
削孔完了後にインナーロッドを引き抜き、保孔管を１本ずつ挿入する。
パイプ挿入時は記入した番号順に挿入し、全数量が挿入できていることを確認する。



Figure：Strainer Pipe Processing

Photo：Perforated Pipes



Photo：Threading / Screw of Pipe Joints

④ Pullout of Casing ケーシング引き抜き

1. After inserting all perforated pipes, pull out casing pipes for drilling.
   保孔管挿入完了後、ケーシングを引き抜く。
2. In case of upward drilling, perforated pipes are often dragged out when casing pipes are pulled out, and connection part of perforated pipes may come off. Thus machine operation must be careful when pulling out the casing.
   上向き削孔の場合、ケーシング引き抜き時に保孔管が一緒に引きずられて出てくることが多いことと、保孔管接続部が外れてしまうことがあるため、ケーシングの引き抜きは慎重に操作しなければならない。
3. Note that if the machine is operated in the direction opposite to the tightening direction of screw joints, connection part of perforated pipes may come off.
   また、接続部のネジ加工の締め付け方向と逆回転で操作した場合、継ぎ手部分が外れてしまうことがあるので注意する。

⑤ Protective pipe 孔口処理

1. To protect the perforated pipes at the ground surface, cover the outlet of the installed perforated pipes with a protective pipe of the size indicated in the design, which are usually one size larger than perforated pipes. .
   To prevent leakage of groundwater discharged from the vicinity of the protective pipes at the outlet, fill the area around protective pipe at the outlet with mortar or the like.
   孔口の保孔管を保護するために、1周り大きいサイズの保護管でカバーする。
   孔口の保護管周辺から排出される地下水が漏れ出さないように、孔口の周囲を硬練りモルタルなどを充填し止水する。
2. Connect the pipe and lead to the drainage channel.
   パイプを接続して排水路へと導く。
3. Attach a cap to the end of the protective pipe so that it can be opened and closed and the perforated pipes can be inspected and maintained (clogging cleaning, etc.).
   保孔管の点検とメンテナンス（目詰まり洗浄など）を可能とするために、保護管の末尾部にはキャップを取り付けて、開閉可能にしておく。
4. Adhere the joint part with a bond so that it will not come off.
   継ぎ手部分は外れないようにボンド等で接着しておく。

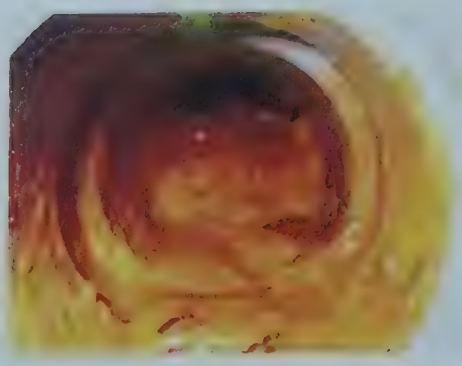
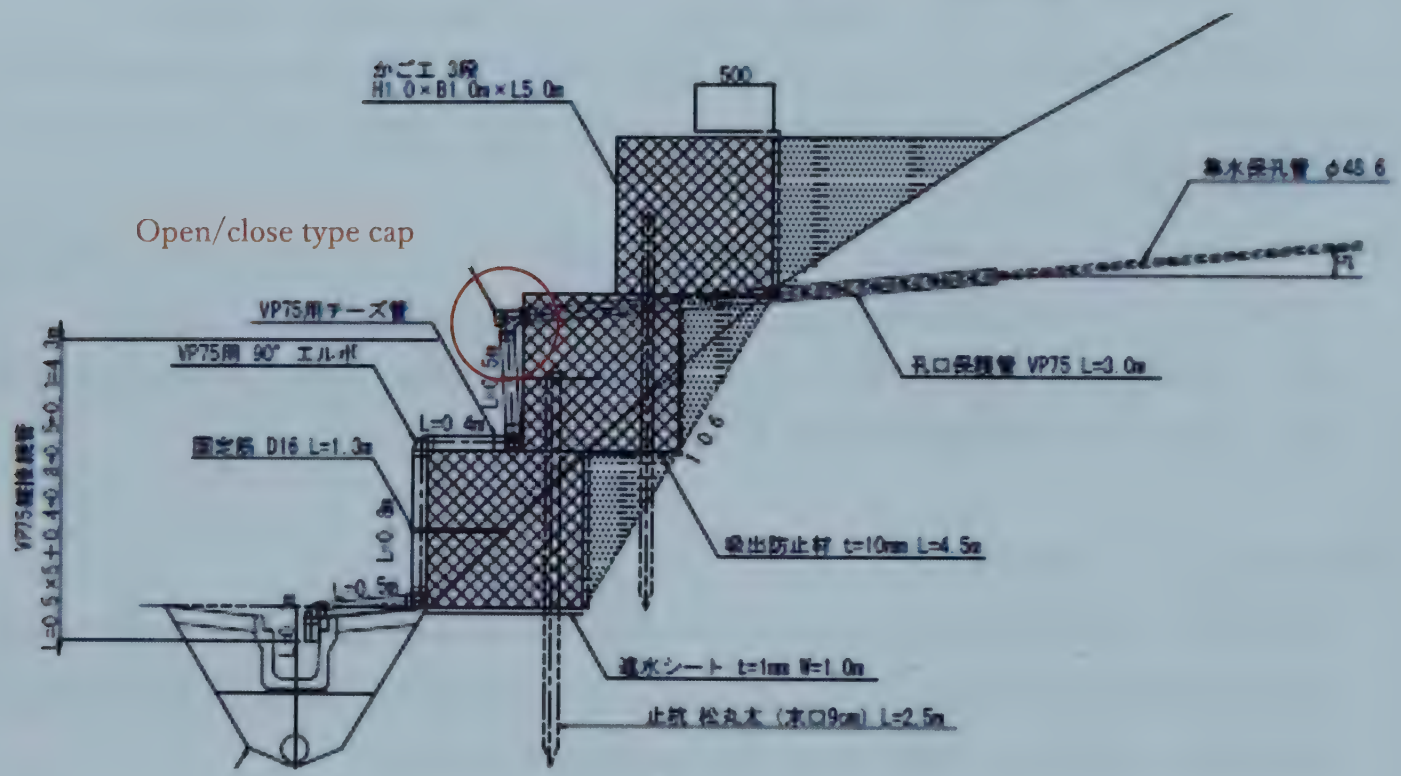


Photo: An example of aged pipe clogging



Figure：Example of Protective Pipes Connection

Photo: Open/Close Type Cap

⑥ Completion 完成

1. After completion, check for groundwater leaks from pipes and outlets.
   If a groundwater leak is found, repair it immediately.
   完成後はパイプや孔口から地下水が漏れ出していないか点検する。地下水の漏れ出

しが見つかった場合は直ちに補修する。

2. It is not the purpose of the works to install the perforated pipes in ground. The purpose of the works is to drain groundwater and lower the groundwater level.
Therefore, if groundwater is not discharged after completion, consult with the Engineer, in case constant groundwater discharge is anticipated.
保孔管を挿入することがこの工法の目的ではない。地下水を排出し、地下水位を下げることがこの工法の目的である。
そのため、完成後に地下水が排出できていない場合は、エンジニアと協議する。

3. If an observation well is available during and after construction within the targeted slope / landslide area, the effect of the Horizontal drainage borings work should be analyzed by comparing the fluctuation of the water level with the volume of water discharged by the works.
もしも同地すべり範囲内に動態観測用の水位計がある場合、水位の変動と同工法により排出される水量を比較し、地下水排除工の効果を分析する。

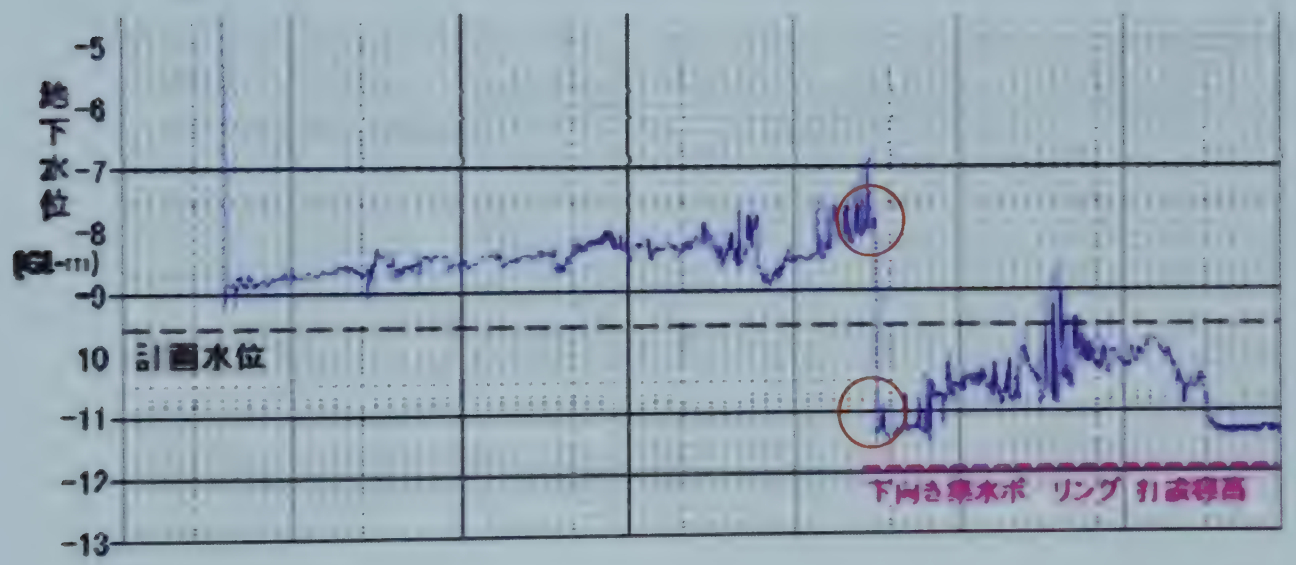


Figure : observation of Groundwater Level

Equipment and Machinery 使用機械

一般的な使用機械は以下の通りである。

Equipment	Specification	Unit
Drilling machine		1
Conveying pomp	Discharge 195L/min～(inverter)	1
Generator	90kva	1
Compressor	To extract slime from boreholes スライム排出用	1
Water pomp	200v 0.4kw	2
Water tank	4m3～	1

Change the type, size and specifications of the drilling machine depending on the site and soil conditions.

現場条件、土質条件により削孔機械の大きさ、仕様を変更する

Temporary and Preparatory Works 仮設計画

The following equipment is generally required for the construction of horizontal drainage boring works.

地下水排除工の施工には以下のような仮設備が必要である。

1. Scaffolding for drilling 削孔用作業床足場

   When using a stationary drilling machine or slid type, a scaffolding for drilling using a steel pipe or H-steel is required.

   定置式削孔機を使用する場合は、単管及びH鋼などを使用した削孔用足場が必要となる。

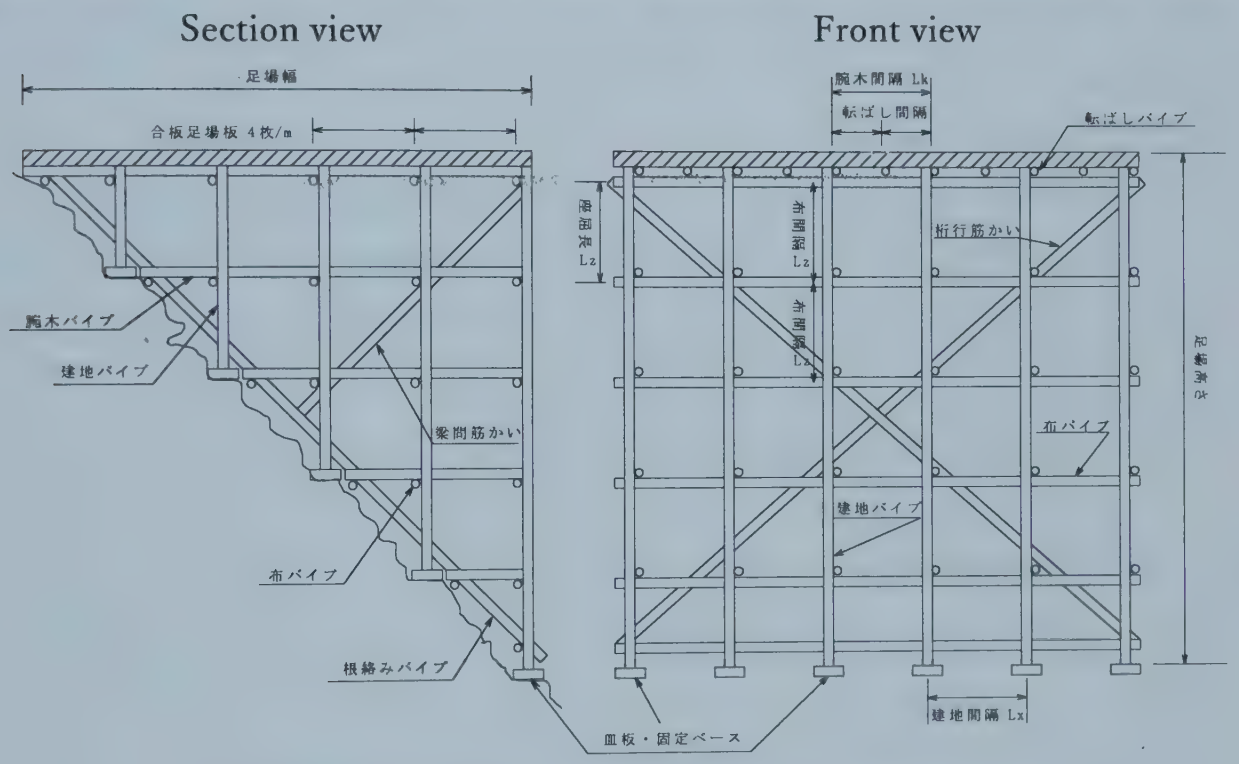


Figure : Example of Assembly for Scaffolding

Work Progress Management and Quality Control 施工管理

Work Progress Management 出来形管理

W1:Drilling Angle 削孔角度

1. Set a drilling machine at the drilling point and check the drilling angle.
   削孔位置に削孔機械を設置して削孔角度を確認する。
2. The frequency of inspection: the drilling angle should be checked at all drilling points, and the tolerance of the error is ±2.5 degrees.
   Since the Horizontal drainage boring is set at an angle of 3 to 10 degrees upward, however, the drilling angle within ± 0.5 degrees from the designed angle or the drilling angle that incorporates expected degree of drilling hole bending is selected at the site.
   削孔角度の確認頻度は全本数であり、誤差の規格値は±2.5度である。
   しかし、排水ボーリングは上向き3度～10度の角度設定であることから、現場では±0.5度以内か、孔曲がりを予測した削孔角度を選択することが多い。
3. Use a digital slant to measure the drilling angle.
   削孔角度の測定はデジタルスラントを使用する。

W2:Drilling Direction 削孔方向

1. When positioning the drilling point, mark the directional survey lines front and behind the drilling point so that it can be confirmed whether there is any deviation in the drilling direction even after the drilling machine is installed.
   削孔ポイントを位置出しする際に、ポイントの前後に削孔方向側線をマーキングして、削孔機械設置後も削孔方向の確認ができるようにしておく。

W3:Drilling Length 削孔長さ

1. After the drilling is completed, measure the length of the drilled bore hole to see if the planned drilling length is maintained.
   削孔完了後、計画された削孔長が確保できているか孔内を測定する。
2. The drilling length can be measured by (1) casing measuring method or (2) measuring rod insertion method.
   削孔長の測定方法としては、①ケーシング検尺方法、②検尺ロッド挿入方法、がある。
3. The frequency of inspection: the drilling length should be checked for all holes. And drilled length tolerances need to be longer than the planned one.
   削孔長さの確認頻度は全本数であり、削孔長計測値は計画された長さ以上が必要となる。

Appendix VIII

1st Technical Transfer Report for Construction Site on NH10

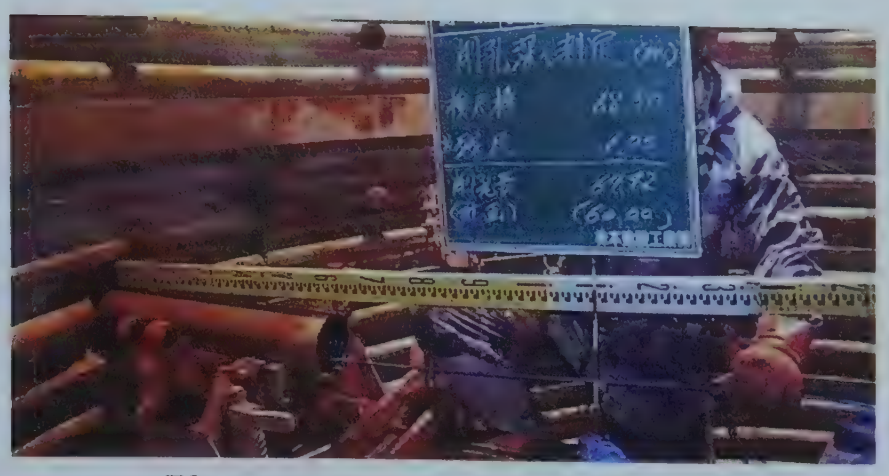


Photo: Drilling Length Inspection

W4: Perforated Pipes Length 保孔管全長

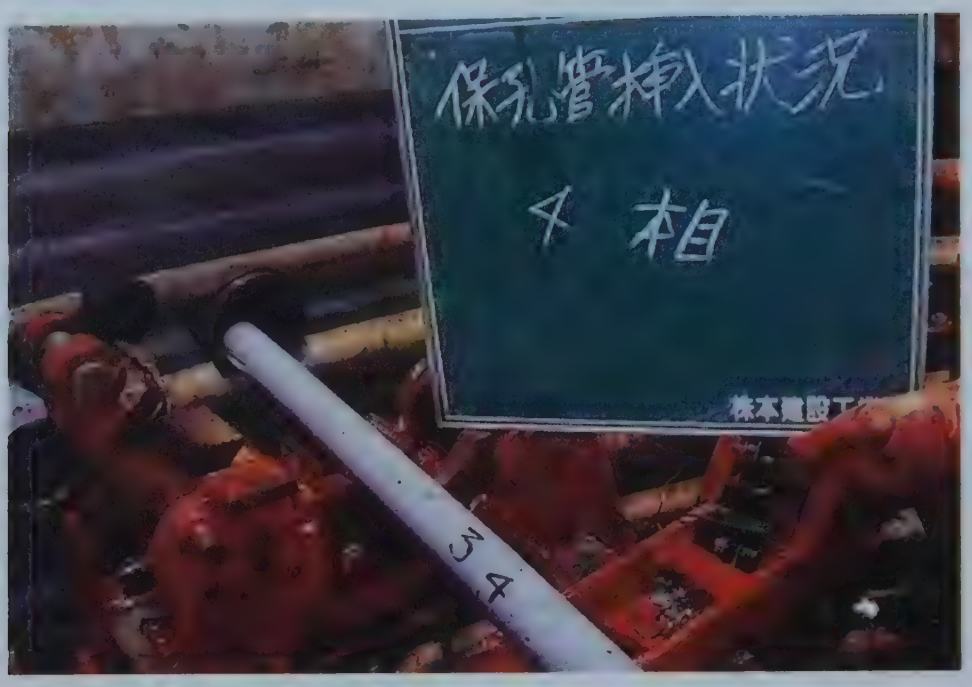
1. Mark the borehole number and the number of the insertion order on the surface of the perforated pipes to be inserted, as shown in the photo below.
   挿入する保孔管には孔番と挿入順序の番号を記載する。
2. Inspect if the length and number of perforated pipes are as planned.
   保孔管の長さ、本数が計画通りになっているか確認する。
3. Frequency of inspection: all pipes.
   確認頻度は全数である。

photo：Inspection for Perforated Pipes Length and Number

W5: Installation Pipes 保孔管挿入

1. To inspect insertion of perforated pipes, check whether the number and the order written on the perforated pipes are inserted one by one in order accordingly.
   保孔管挿入の確認は、１本毎に保孔管に記入しておいた番号と順序通りに挿入されているか確認する。

2. Frequency of inspection：all pipes.
確認頻度は全数である。



photo：Inspection of Inserting Pipes

W6: Protective Pipe Length 保護管長さ測定

1. Measure the length of protective pipe (hard PVC pipe) connected to the perforated pipes at the outlet.
保孔管に接続する保護管（硬質塩ビパイプ）の長さを測定する。
2. Frequency of inspection：all pipes.
測定頻度は全数である。

W7: Measurement of Groundwater Discharge 地下水排出水量測定

1. Check whether groundwater is effectively discharged from the horizontal drainage boring works constructed on site.
地下水排除工の施工によって、効果的に地下水が排出されているかどうか確認する。
2. No tolerance is set for the volume of groundwater discharged from the works. It depends on the underground conditions, such as geological structure, hydrogeological conditions, etc. of the site. As a part of information for confirming the effects of the works, however, measurements of discharge are recommended immediately after construction as well as at two weeks and one month after construction. It is desirable to measure and record the volume of groundwater discharge afterwards.
当該地の地盤条件や地質構造、水文地質条件などにより、排出される水の量に決まりがあるものではないが、地すべり地の挙動を確認するための情報として、施工直後、2週間後、1か月後などの水量を測定して記録することが望ましい。

3. When groundwater discharge is observed, record weather conditions including precipitation as well as groundwater levels in monitoring wells and water levels at nearby ponds / streams in the slope / landslide area.
その際は、天候（雨の場合は降水量）、地すべりエリア内の水位計のデータを同時に記録しておくことも必要である。

photo：Checking Groundwater Discharge

Quality Control 品質管理

Q1:Material 使用材料

1. Prior approval should be obtained from catalogs, copies of mill sheets, etc. before bring-in of materials to be used on site. Only approved materials can be used in the site.
全ての使用材料は試験成績表、カタログなどを監督職員へ示し、承諾を得た後に現場へ搬入する。
2. Inspect the specifications and quantity of the materials brought into the site and confirm that they match the arrival slips.
現場へ搬入した材料については、形状寸法、規格、使用数量、保管状況、品質証明書（ミルシート）について全数量を確認する。

Ministry of Road
Transport and Highways

Capacity Development Project on Highways in Mountainous Regions in India

Japan International
Cooperation Agency

1st technical guidance report

NH10 GAIDARA SLOPE SITE

8th Oct 2021 – 23rd Oct 2021

TANAKA MASAHIDE
of JICA Project Team

Work contents during the period

(8th Oct 2021 – 23rd Oct 2021)

Concrete Crib Works

1. ***Reinforcement Works***
2. ***Form Works***
3. ***Castin Concrete Works***

1. Quality Control

Method Statements

★Issues★

Understanding the following points by inspector, supervisor, contractor, etc.

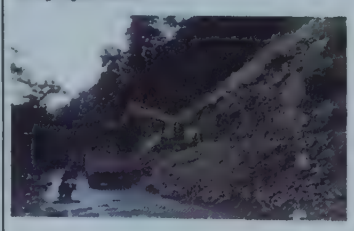
- *construction procedure.*
- *actual construction drawing.*
- *actual construction quantity.*
- *content, frequency and standard value of quality control.*
- *type and quality standard of the materials used in the site.*
- *mixture design for concrete and cement grout.*
- *method and type of temporary construction.*
- *type and number of construction Equipment and the machine layout.*
- *format to inspect such as check sheet*
- *organizational structure*

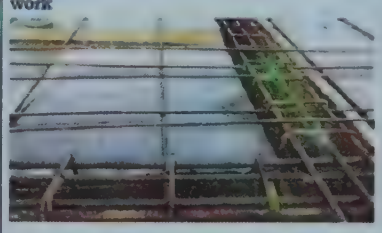
1. Quality Control

Method Statements

★Issues★

Actually approved MS

Methodology of Rock Fall Protection Work	Recommendations by JICA Expert
Site preparation 	• This section is in service, so temporary rockfall trap fences are required during the construction of countermeasures. • If the lower part of the slope is excavated first, the upper part of the slope is likely to be unstable and eventually cause collapse. The cutting or soil removal works, therefore, should start at the top of the slope and then shift downward. The construction procedure starting at the top of slopes and shifting downward is called top-down construction, as shown in Page 3-18 of the Slope Guideline. A construction planning for top-down construction should be well planned for the slope along the road in service, such as those in this section. • During top-down construction of cast-in-situ concrete crib works, horizontal berms should be arranged in certain height intervals, such as every 7 -10 meters high. Berms divide the targeted slope into some sections and treatment of each section should be done separately along top-down construction.
Top drain (crossing channel)	• The idea of placing the top crossing channel is favorable • The surface runoff collected by this top crossing channel should be drained properly to the side ditches along the road, without being drained out to the surface of the unstable slope • On the vertical drain to the side ditches, catch pits should be allocated at appropriate intervals
Top anchoring of Net	• The slope of gravity deformation cannot be stabilized only by the rockfall protection net. Therefore, it is desirable to install ground anchors on the slope in a grid pattern at regular intervals to unite the slope • Anchors should be fixed at stable bedrock under the loosened zone of gravity deformation

5) Spreading of Net on the cutting slope	• Only vertical wire ropes are proposed along with the rockfall protection nets. However, horizontal wire ropes should also be added in a manner to form mesh pattern with the vertical ones. The crossing between vertical and horizontal wire ropes should be fixed by the anchors aforementioned. (It should be noted that the targeted slope is affected by gravity deformation.)
8) Install upper reinforcement Bars for crib work 	• A detailed method statements for cast-in-situ concrete crib works is required. • Arrangement of reinforcement bars for concrete crib works may follow the requirements of retaining wall in India.
9) Shuttering / Frame placing for crib work	• A detailed method statements for cast-in-situ concrete crib works is required. • The cross-sectional area of frame of crib works is small. So, during casting concrete, careful attention should be paid not to form cavities or cobble layers by choked cobbles. Arrangement of reinforcement bars should be well considered to prevent the aforementioned situation.

1. Quality Control

Method Statements

★For improvements★

- Before stringing of construction, contractor makes an MS which is shown on the previous page.
- PMU and consultant check the contents of the submitted MS, and make the contractor revise it if there is a problem in the MS
- The contractor keeps the revision of the MS until the PMU and the consultant approve it.
- All contents of MS will be approved at least 4 weeks before the start of construction. The construction works should not be started without MS approval. It depends on the guideline/standard of the country.

1. Quality Control

Mixture design for concrete

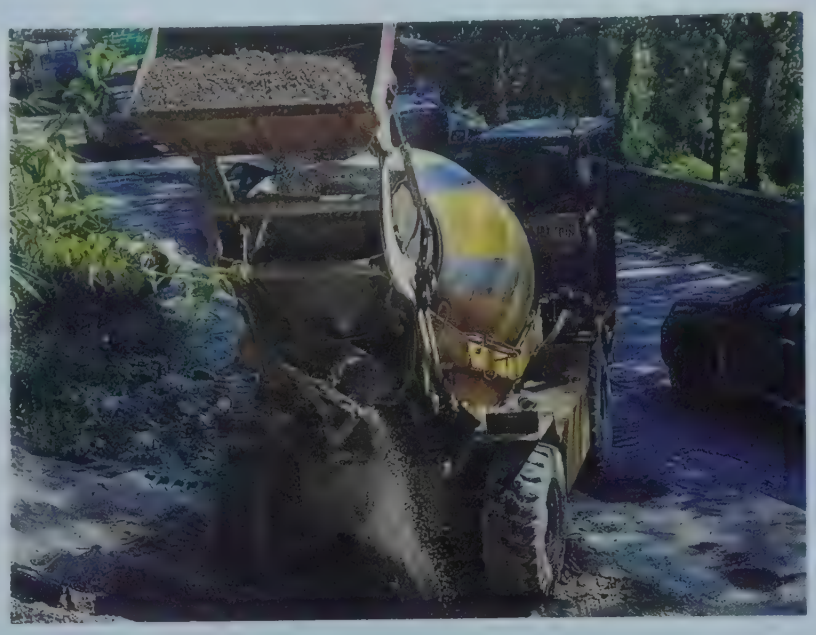
★issues★

- The amounts of aggregate, sand and water were not measured when they mixed concrete materials in the site.
- There is no problem because the cement is counted by the number of bags of 50 kg.

1. Quality Control

Mixture design for concrete

★Point at issue★



1. Quality Control

Mixture design for concrete

★For improvements★

- *Measure the capacity of the basket for aggregate and sand before mixing the concrete.*
- *Fill the basket based on the measured capacity.*
- *Count the number of times when they put it into the agitator.*
- *Check the amount of water every time.*

- *The sand contains water. Percentage of sand surface moisture should be controlled by amount of water considering the daily temperature/humidity condition.*

1. Quality Control

Mixture design of concrete

★For Improvements★

1. Quality Control

Joint position of reinforcing bar

★issues★

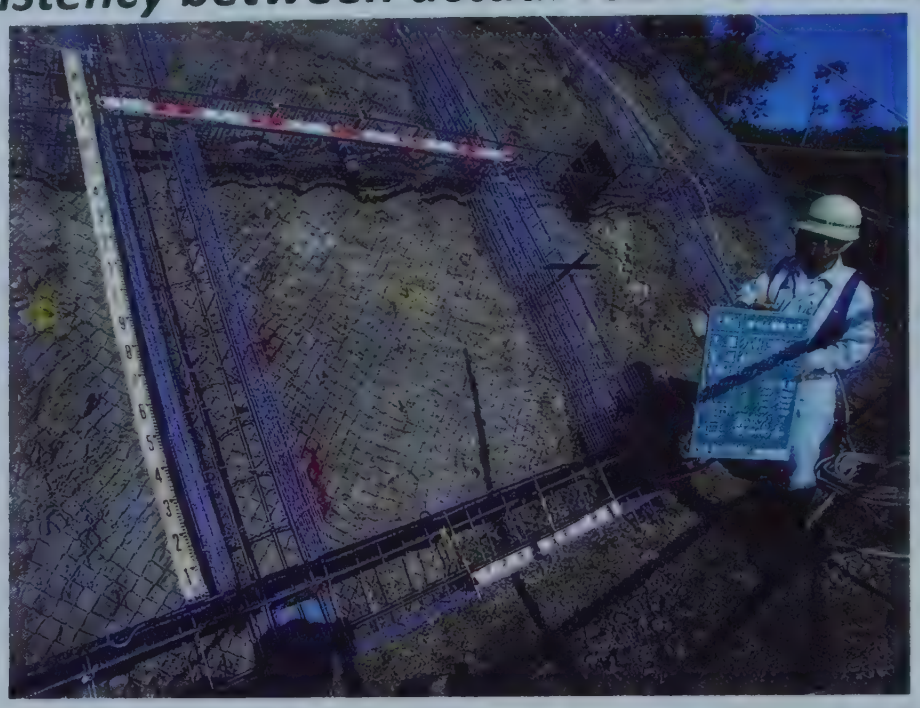
- *All joints were placed near the crossing part of concrete members when the workers assemble the rebar*
- *This makes weak points when earth pressure is loaded to the frame of concrete crib work.*

1. Quality Control

Joint position of reinforcing bar

★For improvements★

- Place the joints of the reinforcing bars in a staggered arrangement of minimum 2m.
- The inspector checks every time before concrete casting to make sure the consistency between actual rebar assembly and drawing.



2. Safety Control

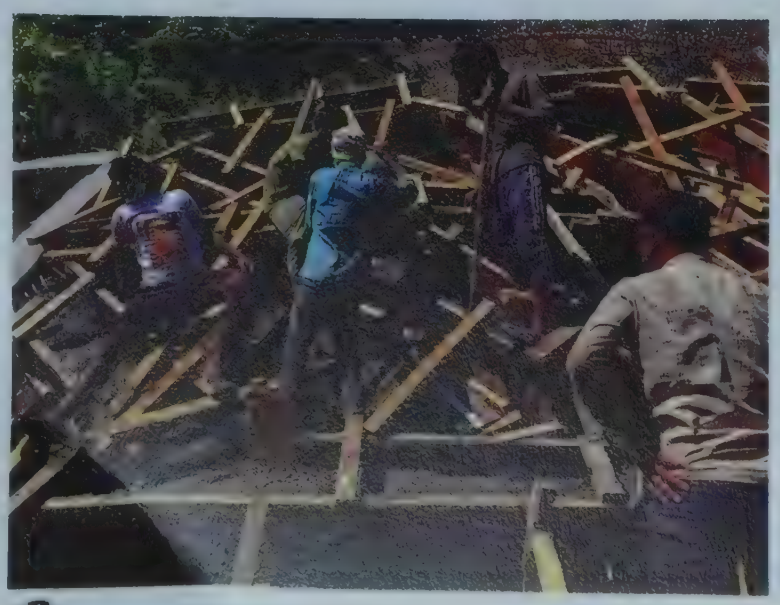
Worker safety

★Issues★

Construction for slope protection contains a dangerous works.

There are risks for workers such as falling from the slope.

Improvement is required.



2. Safety Control

Worker safety

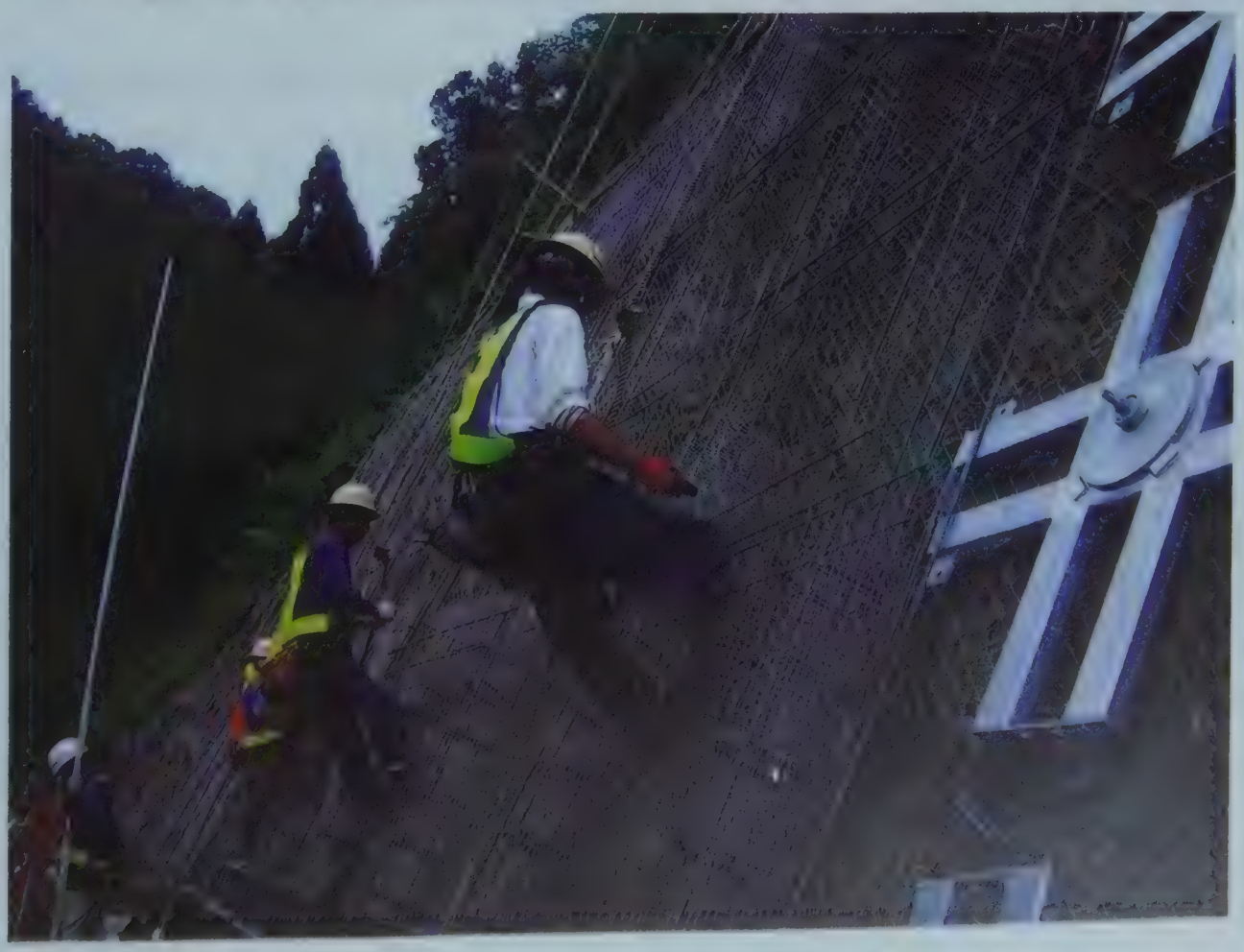
★Way to improve★

- Workers will work on the higher part of the slope.
- Installation of scaffolding or use of harnesses/ropes are required.
- Also, workers should wear the appropriate safety gears to reduce the degree of injury in case of a fall.

2. Safety Control

Worker safety

★For improvements ★use of harnesses/ropes



2. Safety Control

Worker safety

★For improvements ★scaffolding

2. Safety Control

Ensuring the safety of passing vehicles

★Issues★

There are no measures to protect passing vehicles.

If there is a rock fall, it will be a big accident.

I also think that accident prevention measures at night are necessary.

2. Safety Control

Ensuring the safety of passing vehicles

★For improvements★

For example, install such a simple fence.

It can be installed with concrete, H-steel and thick boards.

2. Safety Control

Ensuring the safety of passing vehicles

★For improvements★

Please install a barrier on the shoulder of the temporary access road. It can prevent the rock fall to the road directly .

2. Safety Control

Ensuring the safety of passing vehicles

★Issues★

The construction area is difficult to see at night.

If it is rain at night, the driver's sight might be narrow.

Measures for ensuring the visibility of the construction site are required.

2. Safety Control

Ensuring the safety of passing vehicles

★For improvements★

Place items that are reflected by car lights on the road shoulders and bottoms of slopes.

580×830mm

Schedule of 2nd technical guidance

29th Nov 2021 - 20th Dec 2021